PB85-104552

Nationwide Urban Runoff Program
Tampa, Florida: Phase 3 Final Report
Volume 1. Stormwater Management Plan

Tampa Dept. of Public Works, FL

Prepared for

Environmental Protection Agency, Washington, DC

Oct 83

U.S. Department of Commerce
National Technical Information Service
NTIS

PB85-104552

CITY OF TAMPA

TAMPA, FLORIDA 33602

TAMPA NATIONWIDE URBAN RUNOFF PROGRAM
PHASE III FINAL REPORT
VOLUME I
STORMWATER MANAGEMENT PLAN

October, 1983

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

METCALF & EDDY, INC.
ENGINEERS & PLANNERS
BOSTON / NEW YORK / PALO ALTO / CHICAGO

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO.	2.	3. Recipient's Accession No. PB85 104552
4. Title and Subtitle: Nationwide Urban Runoff Program, Tampa (Florida), Phase III, FINAL REPORT, VOULME I.			5. Report Date: October, 1983
			6.
7. Author(s): Metcalf & Eddy, Inc., Engineers and Planners			8. Performing Organization Rept. No.
9. Performing Organization Name and Address: City of Tampa, Department of Public Works, City Hall Plaza - 4th Floor, Tampa , Florida 33602			10. Project/Task/Work Unit No.
			11. Contract(C) or Grant(G) No. (C) (G) P 004196-01
12. Sponsoring Organization Name and Address: EPA, Office of Water Program Operations, Water Planning Division, 401 M Street, SW, Washington, D C 20460			13. Type of Report & Period Covered: Final , 1979-1983
			14.

15. Supplementary Notes: The five (5) documents produced under the Tampa, Florida, NURP, including this one, are: FINAL SUMMARY REPORT; LOWER HILLSGOROUGH RIVER FINAL REPORT; PHASE II, FINAL REPORT; and PHASE THREE FINAL REPORT, VOLUMES I and II.

16. Abstract (Limit: 200 words)

This volume of the report constitutes the Stormwater Management Plan for the City of Tampa, Florida. It provides the City with an organized approach and specific objectives to effectvely manage its stormwater system, and takes into account both the quantity and the quality of stormwater. Specific objectives in these areas are characterizing the reductions in flows and loads required from the overall study area to eliminate flooding and satisfy receiving waterquality objectives.Practices are recommended to alleviate adverse stormwater impacts. A test basin analysis is included as an example to use in developing specific plans for the area, including needed facilities or controls, and their sizes and costs.

17. Document Analysis a. Descriptors

Hydrology
Urban development
Watersheds
Water pollution
Water resources
Sedimentation
Storms
Regional planning
Urban planning
Civil Engineering

b. Identifiers/Open-Ended Terms

Runoff
Streamflow
Stream pollution
Rainfall
Snowmelt
Soil Erosion
Water Quality
Sediments
Sediment Transport
Suspended Solids
Storm Drains
Urbanization
Land use

c. COSATI Field/Group

18. Availability Statement: No restriction on distribution. Available from National Technical Information Service, Springfield, VA 22161	19. Security Class (This Report) UNCLASSIFIED	21. No. of Pages 126
	20. Security Class (This Page) UNCLASSIFIED	22. Price

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35)

Metcalf & Eddy, Inc.
Engineers & Planners

October 31, 1983

J-7738

Mr. Ron Giovanelli
City of Tampa
Department of Public Works
City Hall Plaza - 4th Floor
Tampa, Florida 33602

Dear Mr. Giovanelli:

In accordance with our Agreement dated July 6, 1981 and subsequent amendments, we are pleased to submit the Phase III Final Report, Stormwater Management Plan for the Tampa Nationwide Urban Runoff Program Study.

Results and findings of the Tampa Nationwide Urban Runoff Program and Receiving Water Study are presented in five (5) final reports including Appendices and fourteen (14) individual task outputs. Included among the final reports are:

1. Phase I Receiving Water Study,
2. Phase II Final Report,
3. Phase III Stormwater Management Plan,
4. Lower Hillsborough River Report, and
5. Final Summary Report.

This study was conducted by Messers. David R. Bingham and Paul N. Singarella and Ms. Katherine O. Thompson of Metcalf & Eddy, under the direction of Dr. Abu M. Z. Alam. Apendices B, C, E and G were prepared from information provided by Priede-Sedgwick staff under the supervision of Mr. Nilo Priede.

We thank you for the cooperation and assistance which you have provided to us during the conduct of the work. It has been a pleasure to work for the City on this important project. We look forward to our continued association with the City of Tampa on future assignments.

Very truly yours,

abu m.z. alam

Abu M.Z. Alam
Associate

AMZA:fpc

PHASE III

CITY OF TAMPA STORMWATER MANAGEMENT PLAN

TABLE OF CONTENTS

	Page
VOLUME I - STORMWATER MANAGEMENT PLAN	
LETTER OF TRANSMITTAL	
TABLE OF CONTENTS	i
LIST OF TABLES	iii
LIST OF FIGURES	v
CHAPTER 1 - INTRODUCTION	1-1
Background	1-1
Stormwater Management Plan (Volume I)	1-1
Stormwater Management Plan Appendices (Volume II)	1-2
Related Studies	1-3
CHAPTER 2 - STORMWATER QUANTITY MANAGEMENT	2-1
General	2-1
Design Storm Development	2-1
Design Storm Flooding	2-7
Development of Flooding Solutions	2-18
Management of Stormwater Quantity	2-32
CHAPTER 3 - STORMWATER QUALITY MANAGEMENT	3-1
General	3-1
Potential Problem Pollutants	3-2
Selection of Design Storm	3-7
Design Storm Pollutant Loads	3-17
Receiving Water Impacts	3-21
Management of Stormwater Quality	3-29
CHAPTER 4 - STORMWATER PERMITS	4-1
General	4-1
Florida Stormwater Permit Regulation	4-1
Impact of Permit on Stormwater Quantity	4-4
Impact of Permit on Stormwater Quality	4-8
Stormwater Management Planning Including Permitting Procedure	4-12
Summary	4-14

METCALF & EDDY

TABLE OF CONTENTS (Continued)

CHAPTER 5 - CASE STUDY - SUNSHINE PARK 5-1

General 5-1
Test Case Area Description 5-2
Preliminary Planning Phase 5-4
Detailed Planning Phase 5-10
Conclusions 5-21

VOLUME II - STORMWATER MANAGEMENT PLAN APPENDICES

APPENDIX A - MATHEMATICAL MODELS

APPENDIX B - DRAINAGE ORDINANCES

APPENDIX C - DESIGN CRITERIA

APPENDIX D - DRAINAGE SYSTEM DESIGN

APPENDIX E - OPERATIONS AND MAINTENANCE

APPENDIX F - LONG-TERM RIVER MONITORING

APPENDIX G - LONG-TERM RAINFALL/RUNOFF MONITORING

LIST OF TABLES

Table		Page
1-1	Other Tampa NURP Study Reports	1-3
2-1	Future No-Action Hydraulic Response of Overall Study Area to Design Storms	2-13
2-2	Summary of 1982 Flooding Complaints by Subarea	2-17
2-3	Flood Mitigation Practices	2-19
2-4	Flooding Solution Requirements for the Five Year Design Storm	2-20
2-5	Flooding Solution Requirements for the Twenty-Five Year Design Storm	2-22
2-6	Flooding Solution Requirements for the Fifty Year Design Storm	2-23
2-7	Summary of Implementation Practice Costs	2-29
2-8	Flood Mitigation Practice Requirements for the Overall Study Area for the Five Year Storm	2-30
2-9	Flood Mitigation Practice Costs for Overall Study Area for the Five Year Storm	2-31
2-10	Scope of Master Drainage Study	2-35
3-1	Comparison of Lower Hillsborough River Water Quality Criteria with Stormwater and River Data	3-3
3-2	Stormwater Quality Parameters of Subbasins Established During Calibration of SWMM	3-11
3-3	Physical Characteristics of the Calibrated Subbasins	3-12
3-4	Design Storm Pollutant Loads by Subarea	3-19
3-5	Summary of WQON Model Inputs for Design Storm Impact Assessment	3-23
3-6	Stormwater Load Reduction Strategies	3-30
3-7	Ranking of Problem Areas by Flooding and Quality	3-33
3-8	Summary of Removal Rates for Various Stormwater Controls	3-35

LIST OF TABLES (Continued)

Table		Page
4-1	Major Components of Florida Stormwater Discharge Regulation	4-2
4-2	Selected Definitions in the Florida Stormwater Permit Regulations	4-3
4-3	Exemptions to Stormwater Permit Regulation	4-4
4-4	Pond Pollutant Removal Rate Due to Flow Detainment	4-10
4-4	Summary of DER Permit Effects on Stormwater Flow and Load Reduction	4-11
5-1	Projected Land Use in Test Case Area	5-4
5-2	Preliminary Requirements and Costs of Alternatives	5-9
5-3	Test Case Area Subcatchment Characteristics	5-12
5-4	Test Case Area Conduit Characteristics	5-14
5-5	Characeristics of Detention Ponds Simulated in Test Case Area	5-17
5-6	Characteristics of Storm Sewer Laterals Simulated in Test Case Area	5-19
5-7	Storm Sewer Works Requirements and Costs for Each Flood Area	5-21

LIST OF FIGURES

Figure		Page
2-1	Cumulative Exceedance Probability of Storm Duration for the Entire Rainfall Record	2-3
2-2	Rainfall Intensity-Duration-Frequency Curves from Tampa International Airport Rainfall Records	2-5
2-3	Derivation of Intensity-Frequency-Duration Relationships Beyond Twenty-Two Years	2-6
2-4	Comparison of Tampa and FDOT 5 Year Intensity-Frequency-Duration Curves	2-8
2-5	Rainfall Hyetograph for the Five Year, Eight Hour Design Storm	2-9
2-6	Rainfall Hyetograph for the Twenty-Five Year, Twenty Hour Design Storm	2-10
2-7	Rainfall Hyetograph for the Fifty Year, Twenty-Four Hour Design Storm	2-11
2-8	Flooding by Subarea for the Design Storm Events	2-14
2-9	Lower Hillsborough River Basin Drainage Subarea Discretization	2-15
2-10	Characteristics of the Generalized Flood Solution	2-24
2-11	Flood Solution Curves for the Design Storm Events	2-25
2-12	Example Use of the 5 Year Flood Solution Curve in Tampa Heights	2-27
2-13	Stormwater Quantity Management Procedure	2-33
3-1	Rainfall Intensity-Frequency-Duration Curves from Tampa International Airport Rainfall Records	3-8
3-2	Rainfall Hyetograph for the One Month, Two Hour Design Storm	3-10
3-3	Site Locations of the Calibrated Subbasins	3-13
3-4	Total Suspended Solids Load for the Design Storms in the North Jesuit Basin	3-15

LIST OF FIGURES (Continued)

Figure		Page
3-5	Mean Total Suspended Solids Concentration for the Design Storms in the North Jesuit Basin	3-16
3-6	TSS Design Storm Runoff, Load Histogram to Lower Hillsborough River from Overall Study Area	3-20
3-7	Initial and Final Fecal Coliform Profiles along River for WQON Simulation	3-26
3-8	Initial and Final BOD Profiles Along River for WQON Simulation	3-27
3-9	Initial and Final DO Profiles Along River for WQON Simulation	3-28
3-10	Stormwater Quantity and Quality Management Procedure	3-35
4-1	Stormwater Management Procedure	4-13
5-1	Location of Test Case Area	5-3
5-2	Five Year Flood Solution Curve Application to Test Case Area	5-7
5-3	Test Case Area Discretization and Storm Sewer Elements with Existing Flood Areas Predicted by SWMM	5-11
5-4	Five Year, Eight Hour Design Storm Hyetograph Used in SWMM Simulations of Test Case	5-15

CHAPTER 1

INTRODUCTION

Background

The City of Tampa is experiencing rapid growth and urbanization, with associated flooding problems. Stormwater runoff is imparting a pollution load to the lower Hillsborough River estuary, which runs through the center of the City. In addition, the State of Florida Department of Environmental Regulation (FDER) enforces a stormwater permitting regulation for new developments, which requires that runoff from the first inch of rainfall be retained on site.

Currently the City does not have integrated stormwater management practices that reflect the scope of the flooding and pollution problems associated with stormwater and current stormwater regulation. In this document, a Stormwater Management Plan (SMP) is developed to serve this purpose.

Stormwater Management Plan (Volume I)

The Stormwater Management Plan (SMP), which constitutes Volume I of this study, provides the City with an organized approach and specific objectives to effectively manage its stormwater system. It takes into account both quantity of stormwater (flooding problems) and quality of stormwater (receiving water pollution problems). Specific objectives in these areas are characterizing the reductions in flows and loads required from the overall study area (the lower Hillsborough River basin) to eliminate flooding and satisfy receiving water

quality objectives. The quantity aspects of stormwater management (flooding) are considered in Chapter 2. The stormwater quality aspects (receiving water impacts) are considered in Chapter 3. The practices recommended to alleviate adverse stormwater impacts are interpreted with respect to State f Florida permitting specifications in Chapter 4. To determine he methods needed to effectively achieve the desired reduction in stormwater flows and loads, a test basin (part of Sunshine Park) is analyzed in detail. This analysis demonstrates techniques to develop specific plans for the area including needed facilities or controls, and their sizes and costs. The procedures followed in the test basin analysis can be used to prepare plans for other basins. The test case is presented in Chapter 5. Volume I was prepared solely by Metcalf & Eddy, Inc.

Stormwater Management Plan Appendices (Volume II)

The City's present approaches to stormwater management including drainage ordinances, design criteria, drainage system design, operation and maintenance and stormwater and receiving water monitoring are reevaluated with respect to the overall objectives of the SMP. Suggested modifications to existing practices will emphasize appropriate design conditions and the coordination of quantity and quality considerations. These areas are covered in the Appendices (Volume II) of the SMP. Several of these appendices (Appendix B on Drainage Ordinances, Appendix C on Design Criteria, Appendix E on Operations and Maintenance, and Appendix G on Long-Term Rainfall/Runoff Monitoring) were prepared

by Priede-Sedgwick, Inc. The others were prepared by Metcalf & Eddy, Inc.

Related Studies

The Stormwater Management Plan represents the output of Phase III of the Tampa Nationwide Urban Runoff Program Studies. In addition to Phase III, the following related reports have already been prepared as part of the Tampa NURP Study:

TABLE 1-1. OTHER TAMPA NURP STUDY REPORTS

Report	Reference
Overall NURP Project Report	(1)
Receiving Water Study	(2)
Phase II Reports	
II.1 Field Monitoring Installations	(3)
II.2 Rainfall Quantity Analysis	(4)
II.3 Rainfall Quality Analysis	(5)
II.4 Runoff Characterization Basin Selection	(6)
II.5 Runoff Characterization Data Collection	(7, 8)
II.6, II.9 and II.10 Analysis of Runoff, Control Testing Data and Projection of Study Area-Wide Loadings	(9)
II.7 Control Testing Basin Selection	(10)
II.8 Control Testing Data Collection	(11,8)
II.12 Quality Assurance/Quality Control Manual	(12)

REFERENCES

1. Metcalf & Eddy, Inc. and Priede-Sedgwick, Inc., Tampa NURP Study, Final Report, October, 1983.

2. Metcalf & Eddy, Inc., Water Quality Assessment of the Lower Hillsborough River, City of Tampa, Tampa Nationwide Urban Runoff Program Study, October, 1983.

3. Priede-Sedgwick, Inc., Field Monitoring Installations, Task II.1, Tampa Nationwide Urban Runoff Program Study, Phase II, February, 1983.

4. Metcalf & Eddy, Inc., Task II.2, Rainfall Quantity Analysis, City of Tampa, Tampa Nationwide Urban Runoff Program Study, Phase II, February, 1983.

5. Priede-Sedgwick, Inc., Precipitation Quantity and Quality Data, Task II.3, Tampa Nationwide Urban Runoff Program Study, Phase II, February, 1983.

6. Priede-Sedgwick, Inc., Runoff Characterization and Watershed Selection, Task II.4, Tampa Nationwide Urban Runoff Program Study, Phase II, December, 1982.

7. Priede-Sedgwick, Inc., Runoff Characterization Water Quality and Flow Data, Task II.5, Tampa Nationwide Urban Runoff Program Study, Phase II, March, 1983.

8. Priede-Sedgwick, Inc., Detailed Event Data, Tasks II.5 and II.8, Tampa Nationwide Urban Runoff Program Study, Phase II, April, 1983.

9. Metcalf & Eddy, Inc., Phase II Report, Task II.6 Runoff Characterization-Data Analysis, Task II.9 Control Testing - Data Analysis, Task II.10 Projection of Study Area-Wide Loadings, City of Tampa, Tampa Nationwide Urban Runoff Program Study, October, 1983.

10. Priede-Sedgwick, Inc., Control Testing Basin Selection, Task II.7, Tampa Nationwide Urban Runoff Program Study, Phase II, June, 1982.

11. Priede-Sedgwick, Inc., Control Testing Water Quality and Flow Data, Task II.8, Tampa Nationwide Urban Runoff Program Study, Phase II, March, 1983.

12. Priede-Sedgwick, Inc., Quality Assurance/Quality Control Procedures Manual, Task II.12, Tampa Nationwide Urban Runoff Program Study, Phase II, December, 1982.

CHAPTER 2

STORMWATER QUANTITY MANAGEMENT

General

The first step in development of a Stormwater Management Plan (SMP) for the City of Tampa is to address the quantity problems associated with stormwater. In this Chapter, existing flooding in the City's lower Hillsborough River drainage basin is assessed, and flood mitigation methods are developed. This analysis includes:

1. Development of design storms for use in flooding assessment,
2. Determination of the extent of flooding resulting from the design storms,
3. Development of flooding solutions, and
4. Development of a stormwater management planning approach to select a final flooding solution.

In subsequent Chapters, the issues of stormwater quality and the State stormwater discharge permit system are addressed.

Design Storm Development

When runoff rate exceeds transport capacity of a drainage system, available storage volume starts to fill up. Once available storage volume is filled, additional runoff may result in flooding. Flooding increases with increasing storm intensity and duration. Storm sewer networks are generally designed to provide different levels of protection depending on the impact of the flooding on the drainage area. For example, storm sewers in

residential areas are most commonly designed for storms with a return period of five years (1). This is because additional flooding damage in residential areas during larger storms is usually proportionately minor. The use of a five year design storm in residential areas is currently being followed in Tampa (2). In commercial, downtown and other high value districts generally higher return periods are used and return periods for these areas can be as high as fifty years (1). When designing storm sewers for a certain return period the occurrence of flooding is limited to those occasions that exceed these events.

For flooding protection in Tampa, design storms with return periods of five, twenty-five and fifty years are developed from the available rainfall record. This record consisted of twenty-two complete years of hourly rainfall measurements made at the Tampa International Airport (3). Storm magnitudes, intensities, and durations were determined for each design storm by analyzing the rainfall record. Details of the duration analysis are presented in Figure 2-1, which plots the percentage of total storms with longer duration versus duration. The percentage of total storms with longer duration is matched with the return period probability to determine design storm duration. For instance, for a five year return period, (which has a 20 percent probability of being equaled or exceeded in any year) the duration is 8 hours. As seen from Figure 2-1, the

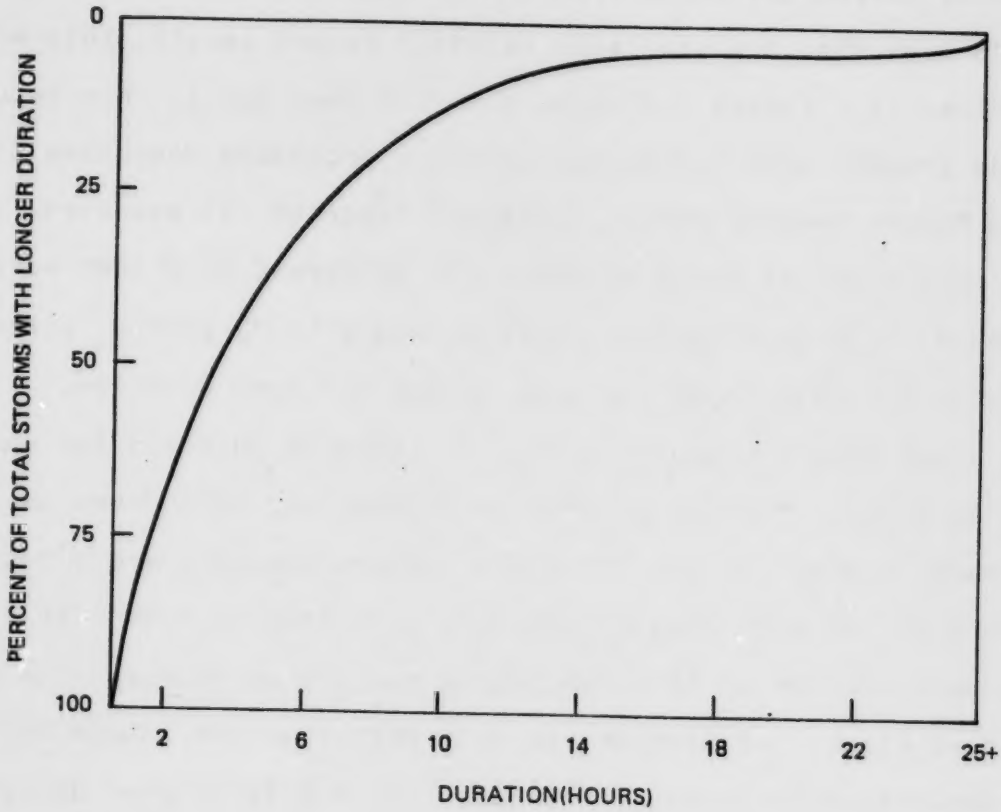


FIG. 2-1 CUMULATIVE EXCEEDANCE PROBABILITY OF STORM DURATION FOR THE ENTIRE RAINFALL RECORD

durations for storms of twenty-five and fifty year return periods are 20 and 24 hours, respectively.

Total rainfall volume and time distribution of the rainfall in Tampa was determined from intensity-duration-frequency curves developed from the rainfall records. For return periods less than the available rainfall record length, this was done directly. Figure 2-2 shows the five year curve. For return periods greater than twenty-two years, a procedure described in United States Weather Bureau Technical Paper 40 (4) was used. Log-normal plots of total rainfall for different durations were developed for return periods ranging from 2 to 22 years. Lines were then fitted through the data points for each duration. These lines were extrapolated from 22 years to 50 years for each storm duration. Results of this extrapolation, which were used to obtain rainfall depths for higher return periods, are shown in Figure 2-3. As seen from Figure 2-3, durations of 1 through 24 hours were considered in establishing the design storms. The six curves of Figure 2-3 provide six data points on the intensity-duration-frequency curves for twenty-five and fifty year design storms. These extrapolated points were used in developing the 25 and 50 year intensity-duration curves presented Figure 2-2.

The City of Tampa currently uses the Florida Department of Transportation (FDOT) intensity-duration-frequency curves (5) in storm sewer design practice. Storm duration is based on the basin time of concentration. Rainfall analysis presented in Figure 2-1 indicates that this duration is excessive for storms

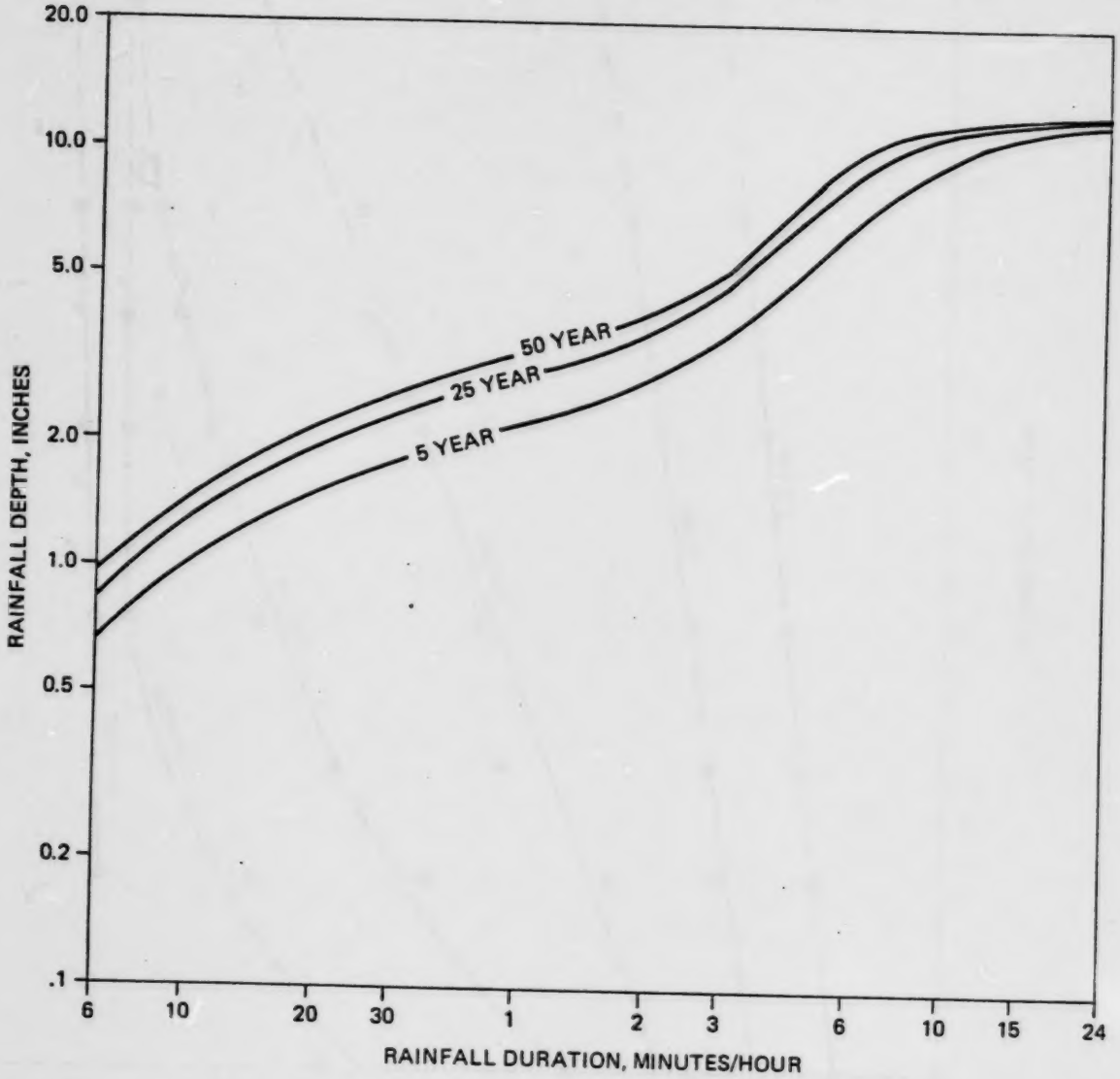


FIG. 2-2 RAINFALL INTENSITY-DURATION-FREQUENCY CURVES FROM TAMPA INTERNATIONAL AIRPORT RAINFALL RECORDS

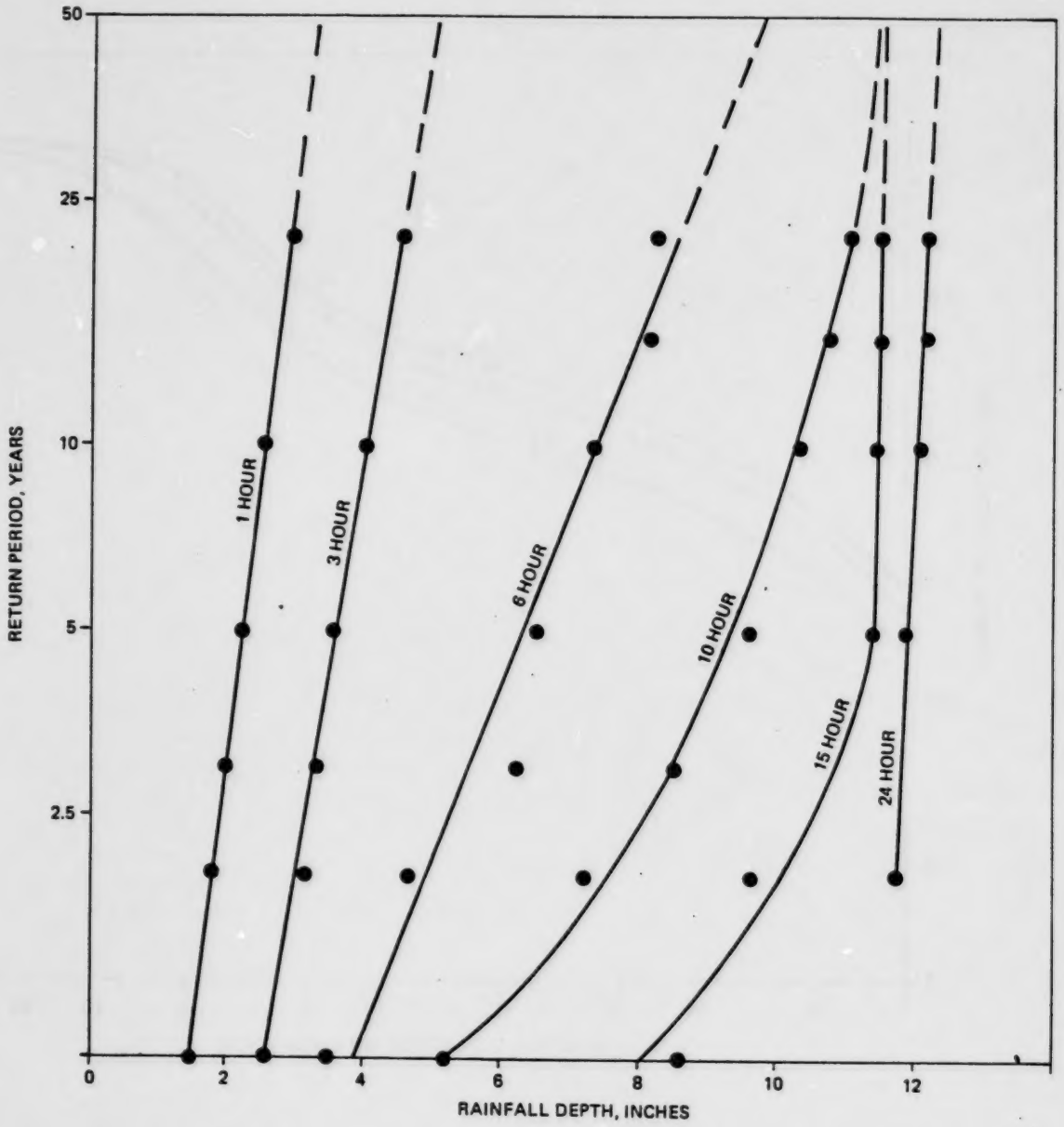


FIG. 2-3 DERIVATION OF INTENSITY-FREQUENCY-DURATION RELATIONSHIPS BEYOND TWENTY-TWO YEARS

less than fifty years. Also, the FDOT design with return appropriate for Tampa. Figure 2-4 compares the curves may ll curve developed in this study with that of the five year from this figure, the FDOT curve underpredicts FDOT. As h in the Tampa vicinity for durations greater than 3 rainfal instance, at 8 hours, FDOT rainfall depth is only 56 hours. percen f that predicted by the Tampa data. Since the Tampa curve s more representative of local conditions, it is used for design storm development in the SMP.

Hyetograph Construction. The average time of peak intensity in Tampa storms was determined to be about 0.5t, where t is the duration of the storm. This value was obtained by analyzing the time to peak intensity of the largest fifty storms in terms of total rainfall. It is assumed that precipitation before and after the peak is identical and symmetrical. A one-hour increment as required by the S/SWMM is used to generate the design storm hyetographs.

Figure 2-5 shows the five-year, eight-hour design storm with a peak one hour intensity of 2.2 inches per hour and a total rainfall of 8.3 inches. The twenty-five year twenty-hour, and fifty-year twenty-four hour design storms are shown in Figures 2-6 and 2-7. These design storms were input to the S/SWMM to quantify flooding impacts in Tampa.

Design Storm Flooding

Modeling Approach. The existing storm sewer system with land use as projected by the City for the year 2000 (6) is

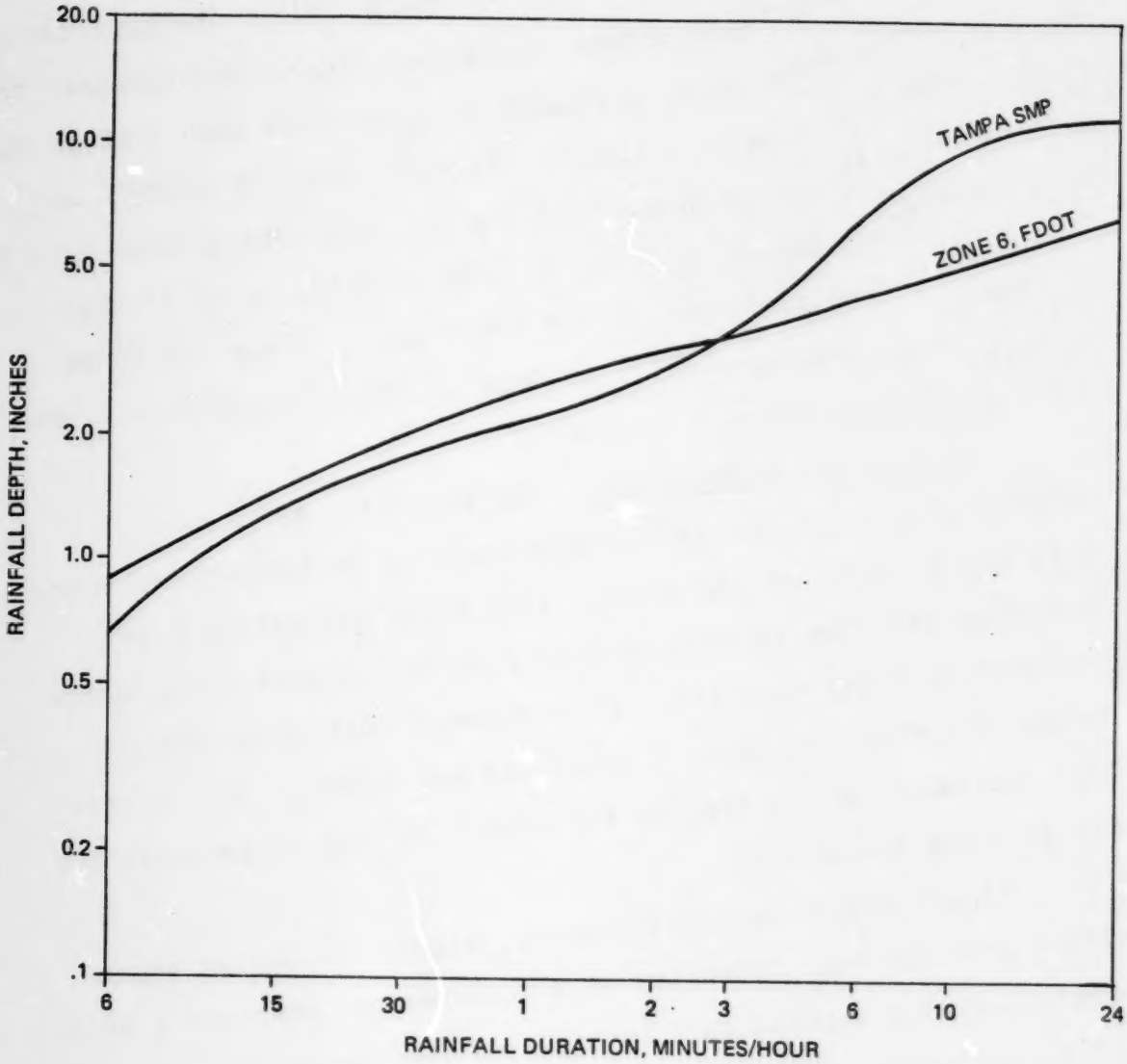


FIG. 2-4 COMPARISON OF TAMPA AND FDOT 5 YEAR INTENSITY-FREQUENCY-DURATION CURVES

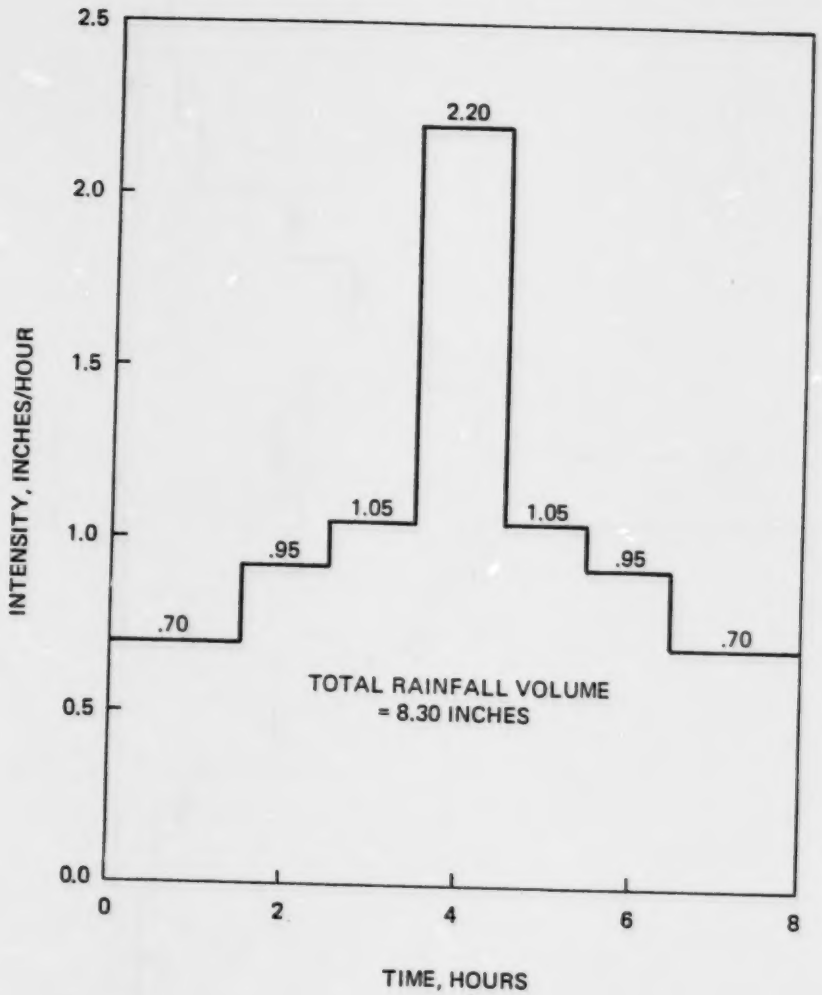


FIG. 2-5 RAINFALL HYETOGRAPH FOR THE FIVE YEAR, EIGHT HOUR DESIGN STORM

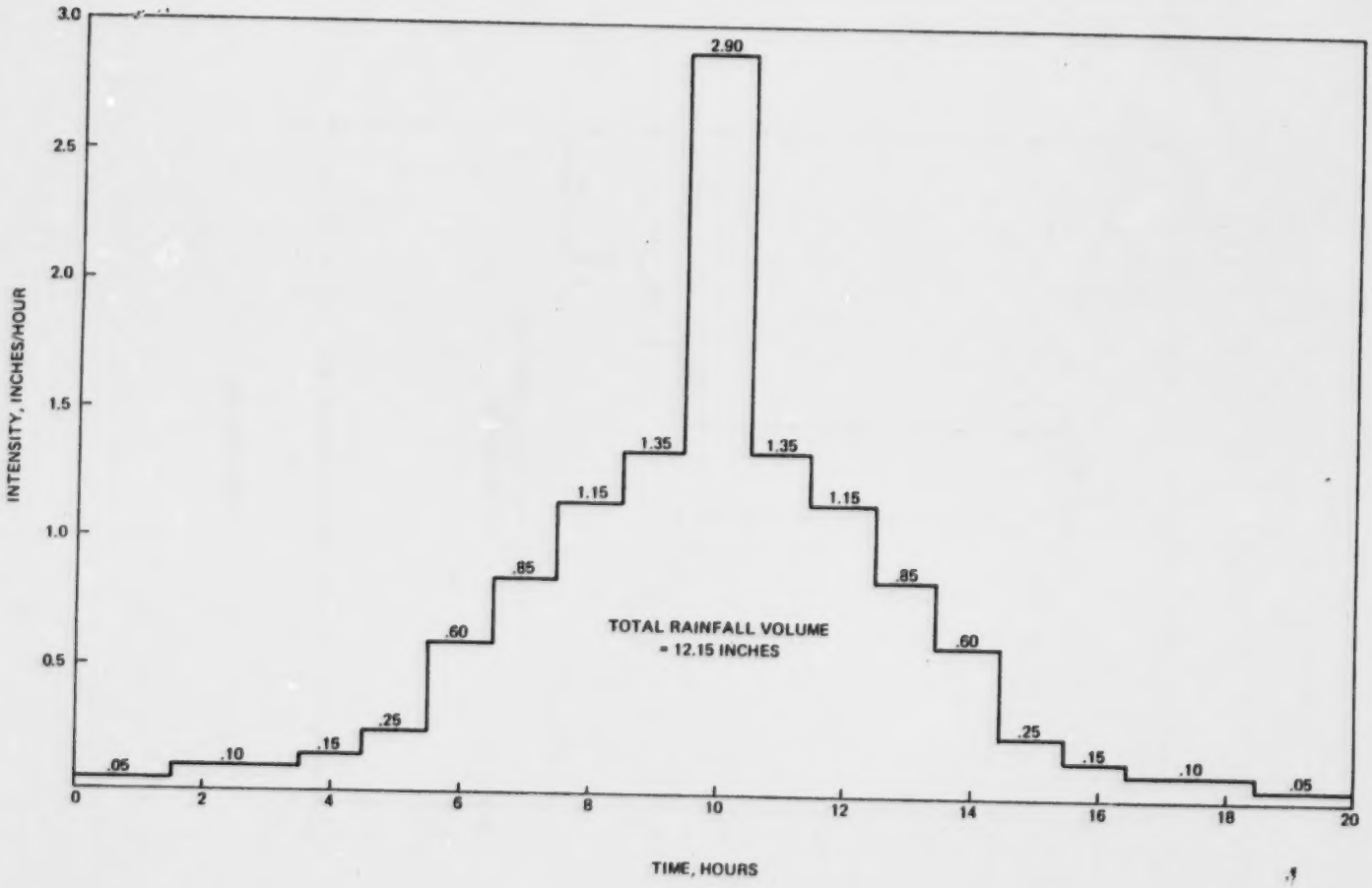


FIG. 2-6 RAINFALL HYETOGRAPH FOR THE TWENTY-FIVE YEAR, TWENTY HOUR DESIGN STORM

METCALF & EDDY

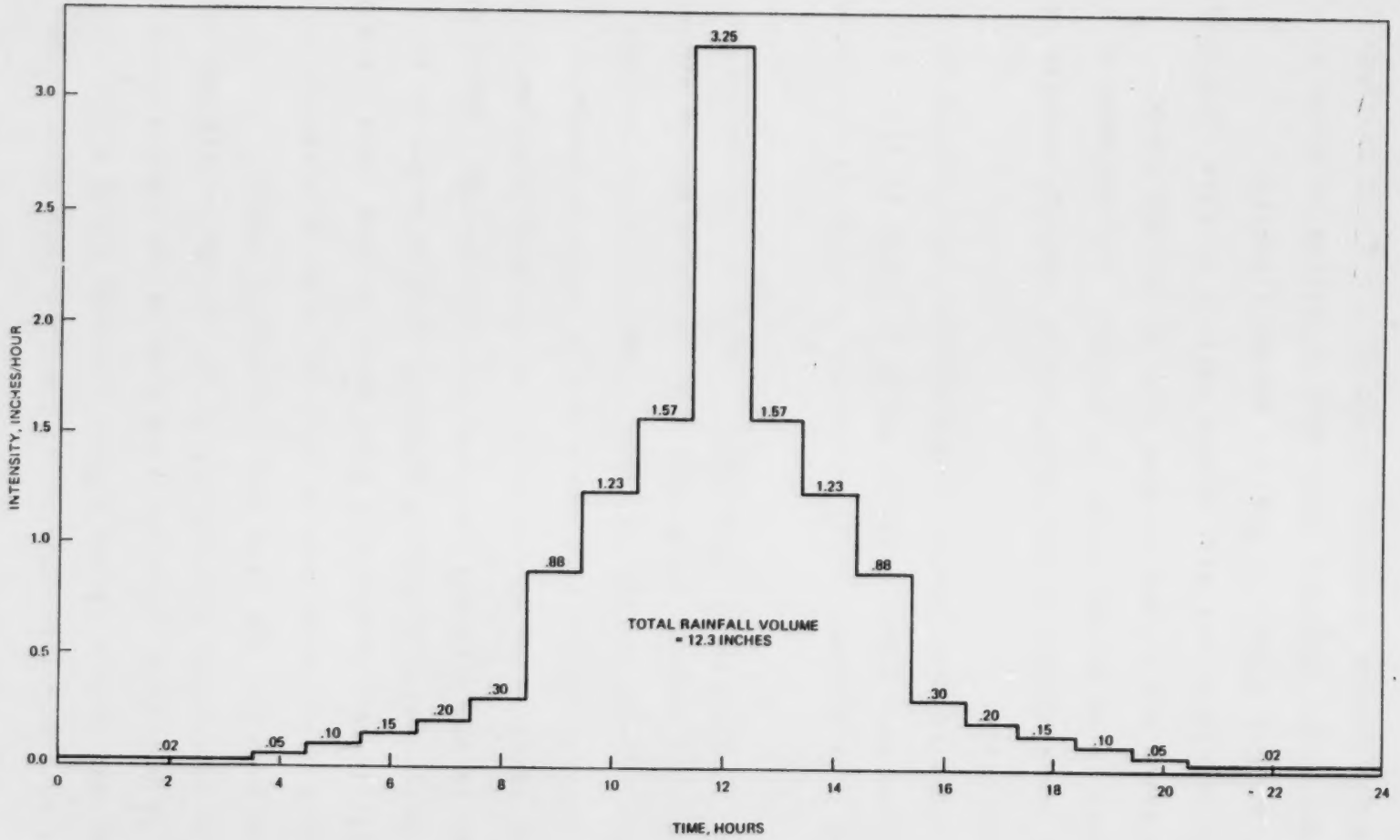


FIG. 2-7 RAINFALL HYETOGRAPH FOR THE FIFTY YEAR, TWENTY-FOUR HOUR DESIGN STORM

defined as the future no-action condition for flood assessment. Flooding was first assessed under this condition in order to ascertain changes needed to prevent future flooding.

The design storms with return periods of five, twenty-five and fifty years are routed through each of the the lower Hillsborough River basins using the S/SWMM. The response of the basins to these storms is then analyzed to identify problem areas within each basin.

A detailed description of the S/SWMM input values for the lower Hillsborough River basin is given in Phase II (7). A summary of this information is presented in Appendix A of this SMP.

Future No-Action Simulations. Table 2-1 characterizes flooding in the overall study area for the three design storms under the future no-action condition. These simulations are based on the assumption that there are no capacity restrictions within the storm sewer system except at the downstream end. This assumption is satisfactory for preliminary planning. Detailed analysis following preliminary planning will be required for solving within-system flooding problems. As seen from Table 2-1 the existing storm sewer system does not provide adequate protection even for the five-year eight-hour storm.

The exceedance probability of the design events decreases an order of magnitude from the five year return period storm to the fifty year return period storm. However fifty year flood volume is slightly less than twice the five year flood volume.

Thus, storm sewer system design providing protection for the five year event will also provide reasonable flood protection during the twenty five and fifty year storm occurrences. This point is graphically illustrated in Figure 2-8 which shows a breakdown of flood volume by subarea for the three design storms. The locations of these subareas within the overall study area are shown in Figure 2-9.

The flood volume/runoff volume ratios in Table 2-1 indicate that over the range of design storms analyzed the percentage of runoff that the storm sewer system can handle decreases slowly as the size of the storm increases.

TABLE 2-1. FUTURE NO-ACTION HYDRAULIC RESPONSE OF OVERALL STUDY AREA TO DESIGN STORMS

Design Return Period (years)	5	25	50
Exceedance Probability (%)	20	4	2
Total Rainfall (inches)	8.6	12.2	12.3
Flood Volume (MG)	451	745	834
Runoff Volume (MG)	1,552	2,272	2,283
$\frac{\text{Flood Volume}}{\text{Runoff Volume}} \times 100$ (%)	29	33	37
Subareas that Flood	10	13	14
$\frac{\text{Flooding Subareas}}{\text{Total Subareas}} \times 100$ (%)	59	76	82

Table 2-1 indicates that flooding is significant during all three design storms with at least 59% of the subareas

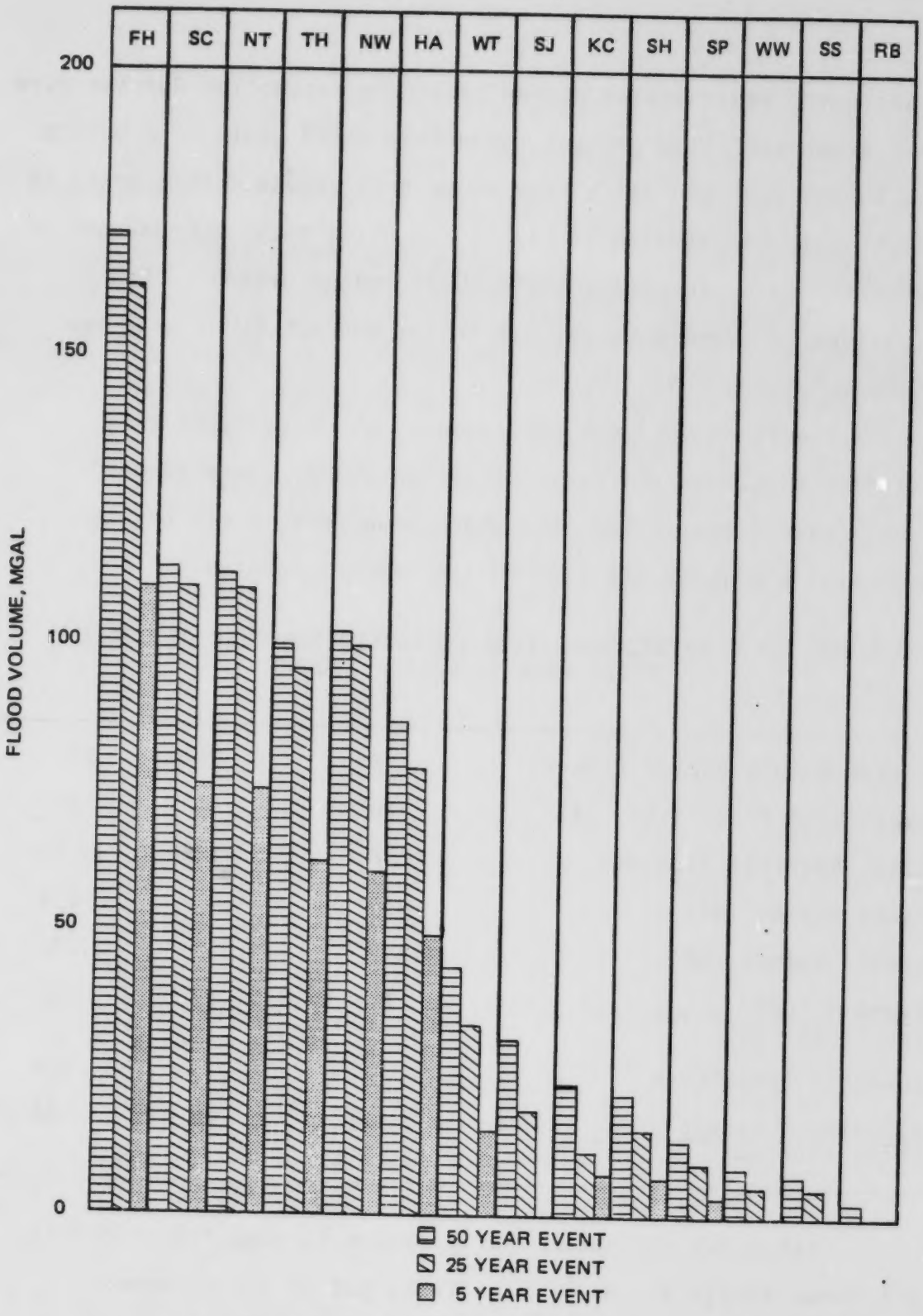


FIG. 2-8 FLOODING BY SUBAREA FOR THE DESIGN STORM EVENTS

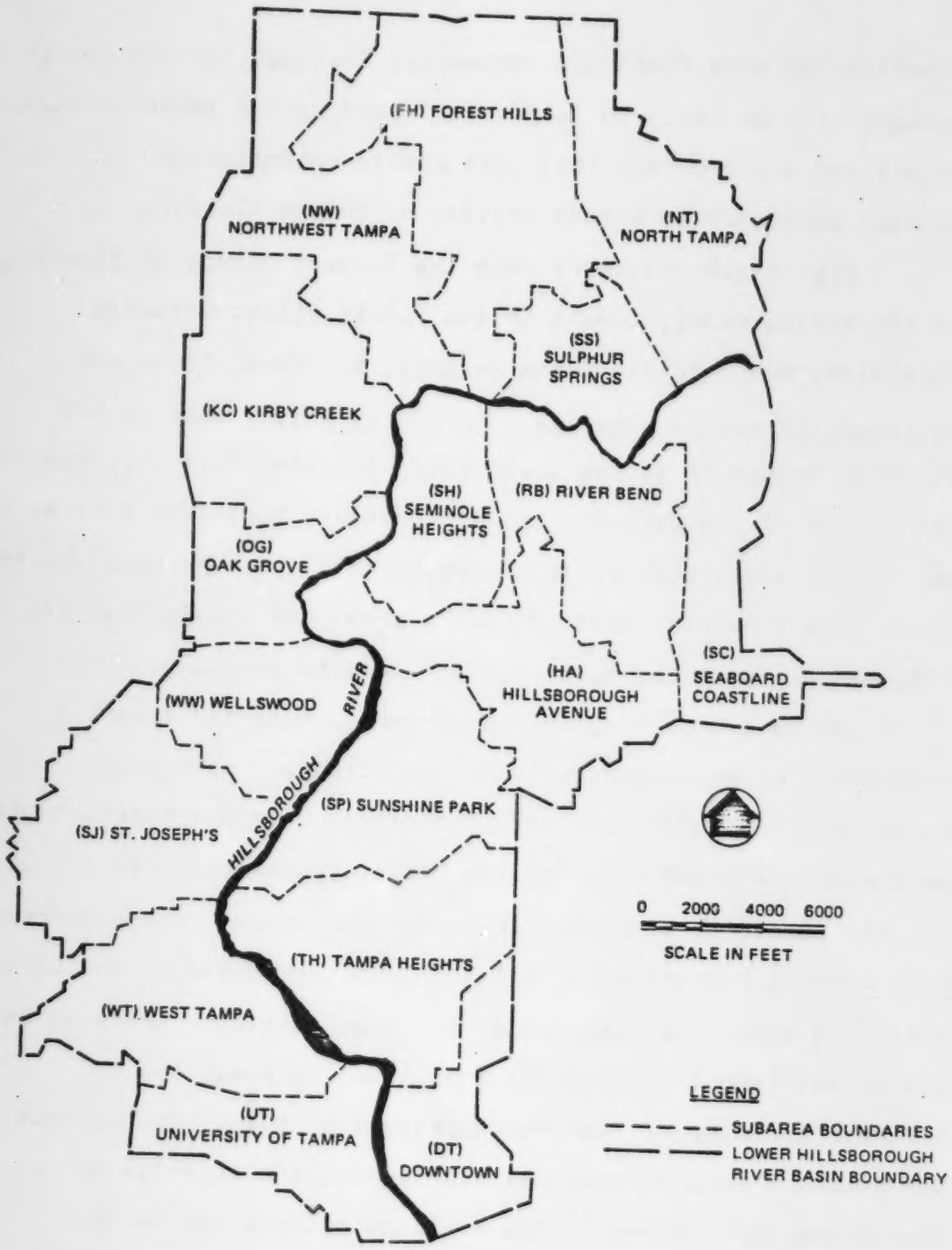


FIG. 2-9 LOWER HILLSBOROUGH RIVER BASIN
DRAINAGE SUBAREA DISCRETIZATION

experiencing some flooding. Extensive drainage network design changes will be required to prevent flooding for these storms. Guidelines are provided that will enable determination of design changes necessary to either prevent or reduce flooding.

Figure 2-8 indicates that the largest amount of flooding, in decreasing order, occurs in the Forest Hills, Seaboard Coastline, North Tampa, Tampa Heights, Northwest Tampa and Hillsborough Avenue subareas. In Forest Hills, some of the overflow volume is pumped to an FDOT retention facility, reducing the volume of floodwaters. As the Seaboard Coastline subarea is not highly developed, it is generally not a problem area for the City. The remaining major flooding areas are residential and will require remedial measures to alleviate flooding.

Table 2-2 is prepared to compare documented flooding problems with results of S/SWMM simulations. Table 2-2 summarizes the documented 1982 flooding complaints attributable to inadequate storm sewer system capacity that resulted in, or had the potential to result in, property damage. These represent only a small percentage of all complaints received by the City in 1982. As seen from this table, the high complaint areas do not always correspond to high flooding areas as predicted by S/SWMM. A number of factors contribute to this. For example, the Sunshine Park subarea has nearly adequate capacity at its downstream end. However, the drainage network has several upstream bottlenecks which cause flooding in the upstream portion of the subarea. The pumping capability of Forest Hills reduces

TABLE 2-2. SUMMARY OF 1982 FLOODING COMPLAINTS BY SUBAREA

Subarea	No. of complaints(1)	% of complaints
SP	36	19.5
KC	24	13.0
WW	22	11.9
OG	20	10.8
HA	17	9.2
TH	15	8.1
WT	12	6.5
FH	9	4.9
RB	8	4.3
SJ, NW, SS	5 each	2.7
NT	3	1.6
SC	2	1.1
DT, SH	1 each	.5
Total	185	

1. Complaints correspond to those attributable to inadequate storm sewer system capacity that resulted or had the potential to result in property damage.

potential floodwaters in that subarea. Although much flooding is predicted at Seaboard Coastline, this subarea is relatively undeveloped; thus few complaints are received.

The City has recently developed a systematic and computerized method to store and evaluate flooding complaint

data. As this data base increases, these assessments can be expanded. The continuation of this effort is certainly an important aspect of stormwater quantity management.

Development of Flooding Solutions

Analyses have been performed to quantify the extent of flooding in the lower Hillsborough River basin for both long-term (7) and design storm conditions. These analyses predict flood volumes and problem subareas. It is the intent of this portion of the Stormwater Management Plan to provide a methodology to characterize existing flooding, evaluate methods to alleviate flooding, and provide preliminary options and cost estimates for flooding solutions.

The development of potential flooding solutions is based upon S/SWMM simulations. It is meant to provide guidelines for stormwater management planning. The analysis will feed into the City's Master Drainage Planning, which will be described later.

Reduction or elimination of flooding requires some combination of the following measures:

1. Expansion of outflow capacity,
2. Enlargement of storage volume,
3. Reduction in runoff volume.

The objective of this analysis is to take into account these strategies in a systematic way for stormwater management planning. The major implementation practices for flood mitigation to be used in planning are presented in Table 2-3.

TABLE 2-3. FLOOD MITIGATION PRACTICES

Flood Mitigation Option	Implementation Practice
Storage Volume Enlargement	Retention/Detention Ponds
Outflow Capacity Expansion	Storm Sewers
Runoff Reduction	Drain Trenches Ditch/Swale Systems

Based on the S/SWMM results, the flood prevention, storage volumes, outflow capacities, and runoff for the five year design storm in each subarea are presented in Table 2-4. Each flood prevention value is calculated as if the other two variables are maintained at existing or projected levels. These levels are the same as those determined in the Tampa NURP Phase II report (7). Changes required in the system to achieve flood prevention are equal to the differences between the flood prevention values and the existing or projected values. Implementation of only one of these three options would control flooding in the various subareas. Similar values for the 25 and 50 year events are presented in Tables 2-5 and 2-6 respectively.

In most cases, a combination of the various mitigation methods listed in Table 2-3 would be implemented as flooding solutions. The values presented in Tables 2-4 through 2-6 can only be used to evaluate individual, single option solutions. For planning purposes, it is desirable to evaluate various

TABLE 2-4. FLOODING SOLUTION REQUIREMENTS FOR THE FIVE YEAR DESIGN STORM

Subarea abbrv.	Existing storage volume (mgal)	Flood prevnt. storage volume (mgal)	Existing outflow capacity[1] (mgal/hr)	Flood prevnt. outflow capacity[1] (mgal/hr)	Pro-jected runoff (mgal)	Flood prevnt. runoff (mgal)
NT	13.7	87.3	1.0	12.1	96.3	22.6
SC	13.3	88.0	2.0	13.6	106.0	29.8
SS	2.7	3.4	10.7	11.3	53.2	50.9
SH	1.9	7.6	7.6	13.8	57.0	36.3
NW	32.9	93.1	1.0	8.6	102.0	41.8
HA	5.2	54.4	6.0	21.9	105.1	42.5
SP	1.8	4.0	18.0	19.8	83.0	75.3
TH	6.8	67.3	4.6	20.7	108.6	38.6
WT	1.9	18.8	9.2	21.2	87.0	42.5
Totals	80.2	424.0	60.1	143.0	798.2	380.3
FH[2]	17.2	125.8	5.9	28.0	178.6	61.8

1. Outflow capacity at downstream end of subarea, i.e. at River outfalls.
2. Assumes no overflow pumping capacity and is therefore not included in totals.

combinations of these controls to determine an optimal solution. In this evaluation, it is also desirable to take costs into account. To accomplish this, a flooding solution curve that contains all possible solutions has been developed.

Flooding Solution Curve. In order to define a general flooding solution curve, both a characteristic storage volume per

METCALF & EDDY

unit area and a characteristic outflow capacity per unit area are specified as:

$$\text{Storage volume per unit area} = \frac{\text{Storage Volume}}{\text{Effective Runoff Area}}$$

$$\text{Outflow capacity per unit area} = \frac{\text{Outflow Capacity}}{\text{Effective Runoff Area}}$$

where,

Effective Runoff Area is defined as Runoff Area x Runoff Coefficient

The flooding solution curve will represent all the storage and outflow combinations that will prevent flooding. Each pair of storage and outflow on the flood solution curve will represent one unique, flood prevention combination of the three factors which may be altered to reduce flooding (i.e. outflow capacity, storage volume and runoff).

The shape and features of the flood solution curve are detailed in Figure 2-10. The shaded area under the curve represents combinations of storage volume and outflow capacity that would cause flooding in a specific subarea.

The flooding solution curves shown in Figure 2-11 were developed from the S/SWMM outputs for all the subareas in the lower Hillsborough River for the three design storm events. Each solution required specification of the following for each subarea:

1. Flood Prevention Storage Volume,
2. Existing Outflow Capacity,
3. Effective Runoff Area,

TABLE 2-5. FLOODING SOLUTION REQUIREMENTS
FOR THE TWENTY-FIVE YEAR DESIGN STORM

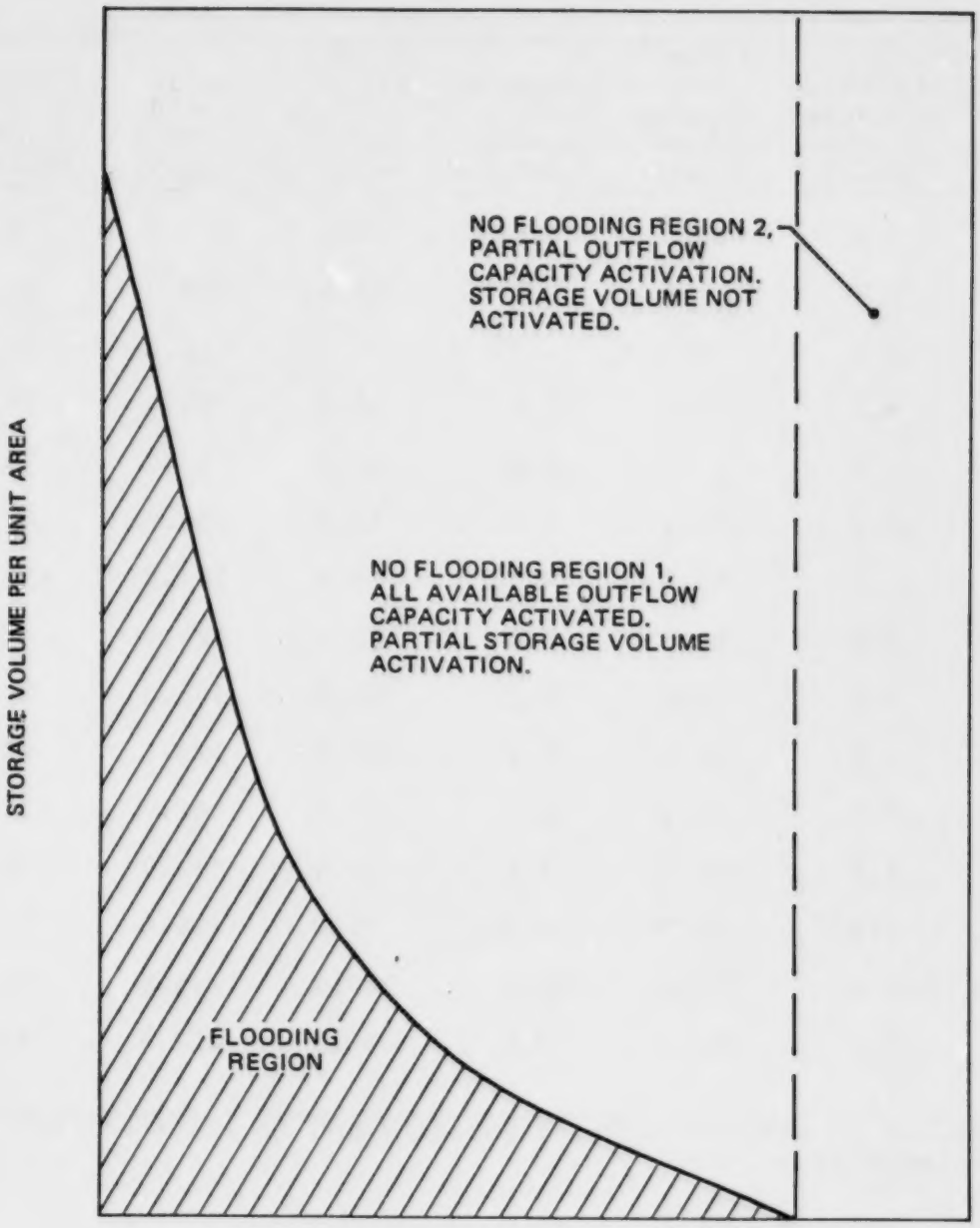
Subarea abbrv.	Existing storage volume (mgal)	Flood prevnt. storage volume (mgal)	Existing outflow capacity (mgal/hr)	Flood prevent. outflow capacity (mgal/hr)	Proj-ected runoff (mgal)	Flood prevnt. runoff (mgal)
NT	13.7	122.3	1.0	18.8	141.0	25.4
SC	13.3	122.3	2.0	22.2	155.2	34.8
SS	2.7	7.9	10.7	16.3	77.9	56.3
SH	1.9	16.5	7.6	18.8	83.5	39.8
NW	32.9	130.6	1.0	13.1	149.3	47.1
KC	17.8	28.9	14.1	19.0	153.0	130.6
HA	5.2	83.0	6.0	31.5	153.8	45.8
WW	2.5	7.5	18.2	24.3	107.7	85.7
SP	1.8	11.0	18.0	27.0	121.6	81.9
SJ	17.4	35.5	13.9	20.6	161.9	128.1
TH	6.8	100.0	4.6	30.6	159.0	45.9
WT	1.9	36.6	9.2	29.0	127.4	47.4
Totals	117.2	702.2	106.3	271.2	1591.3	769.1
FH[1]	17.2	179.6	5.87	44.3	261.5	70.4

1. Assumes no overflow pumping capacity and is therefore not included in totals.

TABLE 2-6. FLOODING SOLUTION REQUIREMENTS FOR THE FIFTY YEAR DESIGN STORM

Subarea abbrv.	Existing storage volume (mgal)	Flood prevnt. storage volume (mgal)	Existing outflow capacity (mgal/hr)	Flood prevnt. outflow capacity (mgal/hr)	Proj-ected runoff (mgal)	Flood prevnt. runoff (mgal)
NT	13.7	125.5	1.0	22.6	141.7	23.9
SC	13.3	126.3	2.0	26.3	156.0	31.5
SS	2.7	10.1	10.7	18.2	78.2	49.9
RB	2.2	5.7	27.4	30.8	124.5	103.6
SH	1.9	23.1	7.6	20.7	83.9	36.0
NW	32.9	133.9	1.0	15.8	150.0	45.6
KC	17.8	40.8	14.1	23.7	153.8	115.6
HA	5.2	92.6	6.0	35.1	154.6	42.6
WW	2.5	10.6	18.2	26.9	108.2	77.3
SP	1.8	14.5	18.0	29.9	122.1	73.8
SJ	7.4	48.2	13.9	25.0	162.7	111.8
TH	6.8	106.7	4.6	34.4	159.8	39.1
WT	1.9	46.6	9.2	32.0	128.0	41.7
Totals	119.4	784.6	133.7	341.4	1723.5	792.4
FH[1]	17.2	188.4	5.9	50.5	262.8	64.6

1. Assumes no overflow pumping capacity and is therefore not included in totals.


NO FLOODING REGION 2, PARTIAL OUTFLOW CAPACITY ACTIVATION. STORAGE VOLUME NOT ACTIVATED.
NO FLOODING REGION 1, ALL AVAILABLE OUTFLOW CAPACITY ACTIVATED. PARTIAL STORAGE VOLUME ACTIVATION.
FLOODING REGION
STORAGE VOLUME PER UNIT AREA
OUTFLOW CAPACITY PER UNIT AREA

NOTE: FLOOD SOLUTION CURVE CORRESPONDS TO 5 YEAR SOLUTION

FIG. 2-10 CHARACTERISTICS OF THE GENERALIZED FLOOD SOLUTION

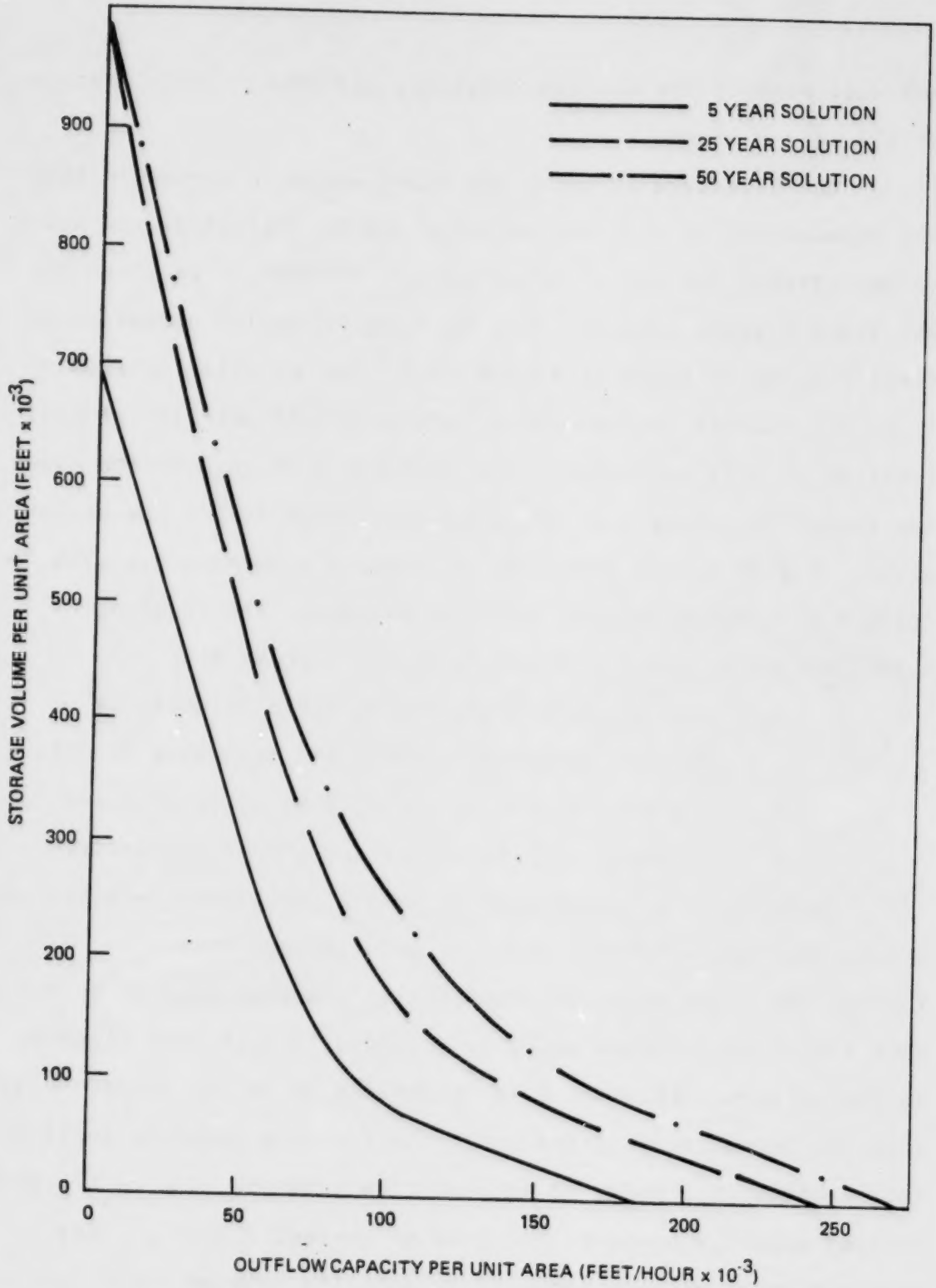


FIG. 2-11 FLOOD SOLUTION CURVES FOR THE DESIGN STORM EVENTS

and knowledge of the maximum intensity and total rainfall volume of the design storm.

The advantage of using the flood solution curves is that any combination of flooding controls can be evaluated. In order to demonstrate the use of these curves, an example is given for the Tampa Heights subarea. The existing no-action condition in Tampa Heights is shown in Figure 2-12. The existing outflow capacity, storage volume, and effective runoff area define the location of this condition. The location lies in a region under the curve indicating that flooding will occur during the design event. The no-action point can be used to determine how best to solve the flooding problem in Tampa Heights. The flooding solutions lie within two distinct bounds. These are:

1. Increasing storage volume while maintaining existing outflow capacity and projected runoff.
2. Increasing outflow capacity while maintaining existing storage volume and projected runoff.

Solution 1 is determined by moving vertically from the no-action position until the flooding solution curve is intersected. The point of intersection provides data as to how much additional storage would be required to eliminate flooding in the subarea. Solution 2 is determined by moving horizontally from the no-action position until the flooding solution curve is intersected. The point of intersection provides data on how much outflow capacity would be required to prevent flooding. Any combination of storage and outflow capacity between these two

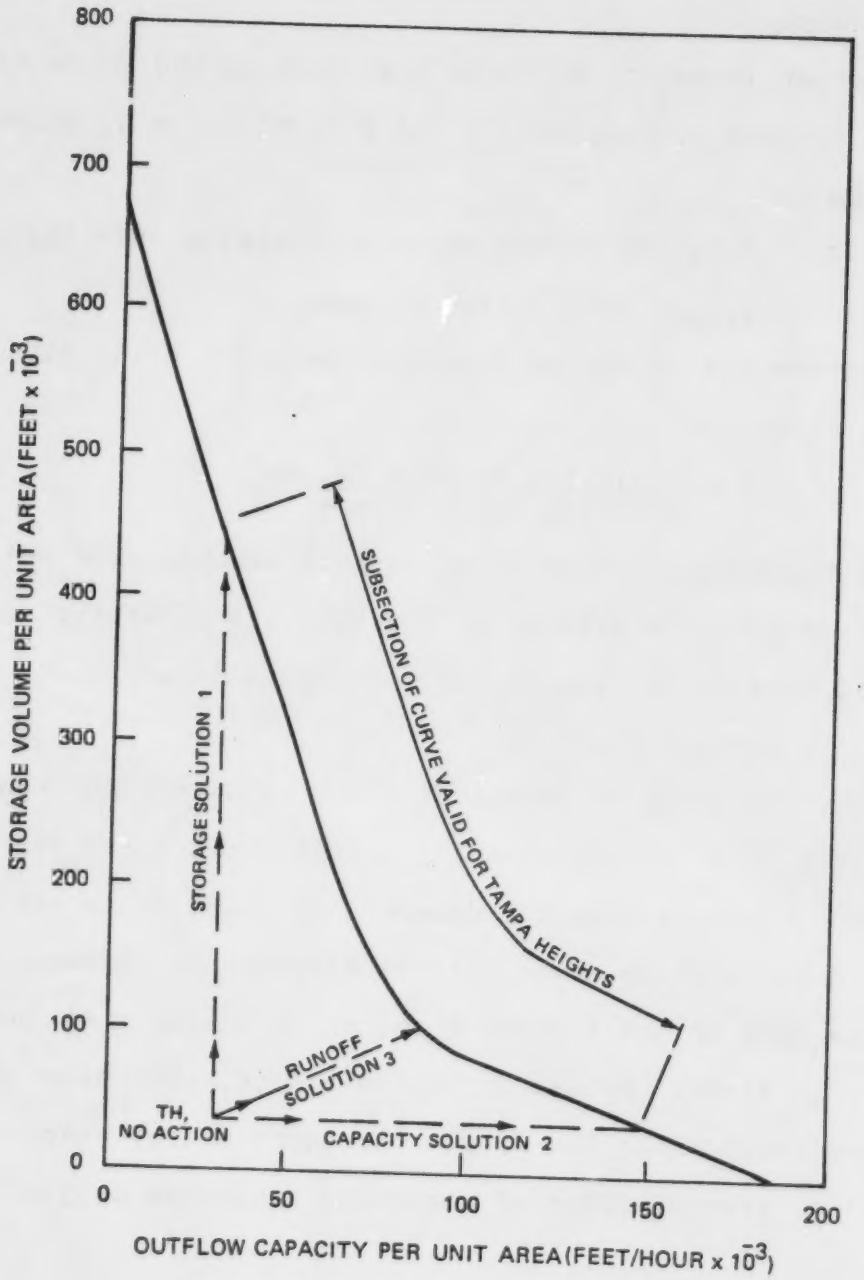


FIG. 2-12 TYPICAL USE OF THE 5 YEAR FLOOD SOLUTION CURVE IN TAMPA HEIGHTS

points along the flooding solution curve would eliminate flooding in Tampa Heights.

Another important solution that lies on the subsection of the curve bounded by Solutions 1 and 2 is Solution 3, which corresponds to:

3. Reducing runoff while maintaining existing storage volume and outflow capacity.

This is determined by moving from the no-action point with the following slope:

$$m = \frac{\text{Existing Storage Volume}}{\text{Existing Outflow Capacity}},$$

until the flooding solution curve is intersected. The point of intersection provides data as to how much the effective runoff area would have to be reduced, by controls such as drain trenches, to prevent flooding.

Once the range of potential flood solutions has been defined, the costs of implementing specific solutions can be estimated. Criteria used for determining these costs are shown in Table 2-7. A storm sewer lateral diameter of 54 inches was assumed as this is the typical diameter of outfalls to the lower Hillsborough River. Unless otherwise stated, the costs in this table were provided by the City. The costs are site-specific and are meant to provide order of magnitude estimates only.

TABLE 2-7. SUMMARY OF IMPLEMENTATION PRACTICE COSTS

Implementation practice	Capital unit costs	Annual O&M ($)
Storm sewer laterals	100/lin. ft.(1)	Not Available
Storm sewer manholes	814 each	Not Available
Storm sewer inlets	1,800 each	Not Avaialble
Drain trenches	3,170/drain trench(2)	26.70/drain trench
Ditch systems/swales	5.69/lin. ft.	.08/lin. ft.
Detention/Retention Ponds	25,000/acre(3) (land acquisition) 21,800 acre(4) (excavation) 2,900/acre(5) (fencing) 12,600/acre (sodding)	136/acre (mowing)

1. Assumes a diameter of 54 inches.
2. Assumes 150 ft., City of Tampa uniform trench.
3. Based on appraisals made in River Grove subarea for a different study by a City approved appraiser.
4. Assumes 3 ft. of excavation.
5. Assumes average pond surface area of 3 acres.

For planning purposes, the implementation needs and costs of stormwater controls can be estimated on a subarea or an area-wide basis. As an illustration, this procedure is used for the entire study area. In accordance with the Phase II results (7), it is assumed that 150 feet of drain trench or ditch will remove 30 percent of the runoff from 2 acres of drainage area. The total amount of retention/detention pond acreage, storm sewer works, ditches or drain trenches required to alleviate flooding for the five year storm in the lower Hillsborough River basin are as follows:

TABLE 2-8. FLOOD MITIGATION PRACTICE REQUIREMENTS FOR THE OVERALL STUDY AREA FOR THE FIVE YEAR STORM

Practice	Requirement
Retention/Detention Pond(1)	352 acres (0.55 sq. mi.)
Storm Sewer Works	
Storm sewer outfalls	29(2)
Storm sewer laterals	116,000 feet(3)
Storm sewer manholes	232(4)
Storm sewer inlets	232(4)
Drain Trenches(5)	12,500
Ditches	1.9 million feet

1. Assumes 3 feet of storage available above outflow weir, or about one million gallons storage/acre.
2. Assumes a diameter of 54 inches.
3. Assumes a 4,000 ft. approach to each outfall.
4. Assumes 1 manhole and inlet per 500 ft. pipe.
5. Assumes 150 ft. of standard trench per unit.

Combining the requirements of each solution (Table 2-8) with the costs of each practice (Table 2-7), the total estimated cost of each single practice solution for the overall study area has been determined. These costs are presented in Table 2-9. The generalized costs presented in Table 2-9 were generated to facilitate an understanding of the relative merits of the various implementation practices. More specific information can be produced by applying this procedure to the subareas. Such calculations clarify the relative merits of the flood mitigation practices and facilitate the determination of the most cost-effective control combination. However, each subarea will have physical limitations on control implementation which must be incorporated in the analysis in order to arrive at a realistic solution to the flooding problem in the subarea.

TABLE 2-9. FLOOD MITIGATION PRACTICE COSTS FOR OVERALL STUDY AREA FOR THE FIVE YEAR STORM

Practice	Cost	
	Construction ($ x 10^6)	Annual O&M ($)
Retention/Detention Ponds	21.9	47,900
Storm Sewer Laterals	12.2	Not Available
Drain Trenches	39.6	334,000
Ditches	10.8	152,000

Management of Stormwater Quantity

Figure 2-13 illustrates a step-by-step procedure which incorporates the aspects of stormwater quantity management presented in this Chapter as well as in the Tampa NURP Phase II Report (7). The procedure shown in Figure 2-13 for stormwater quantity management will be expanded upon in Chapter 3 to include stormwater quality management, and in Chapter 4 to include stormwater permitting requirements. The approach involves a preliminary planning phase as well as a detailed planning phase. The preliminary planning phase can be conducted by the City staff using the methodology presented in this report. The detailed planning phase is comparable, with some revisions, to the City's master drainage planning effort. Each step is discussed in the following paragraphs. An application of these concepts is given as a test case in Chapter 5 of this Report for one of the lower Hillsborough subareas.

Preliminary Planning Phase. This phase of stormwater quantity management planning consists of incorporating information included in the SMP for a particular subarea and developing a preliminary range of potential solutions and order of magnitude costs for flooding solutions.

Based on the flooding complaint data and the results of the S/SWMM analysis (i.e. Tables 2-1 through 2-5), the problem subareas have been identified. These can be ranked by the City in terms of their priority for problem correction. When flooding is projected for the 5, 25 or 50 year design events, Tables 2-3

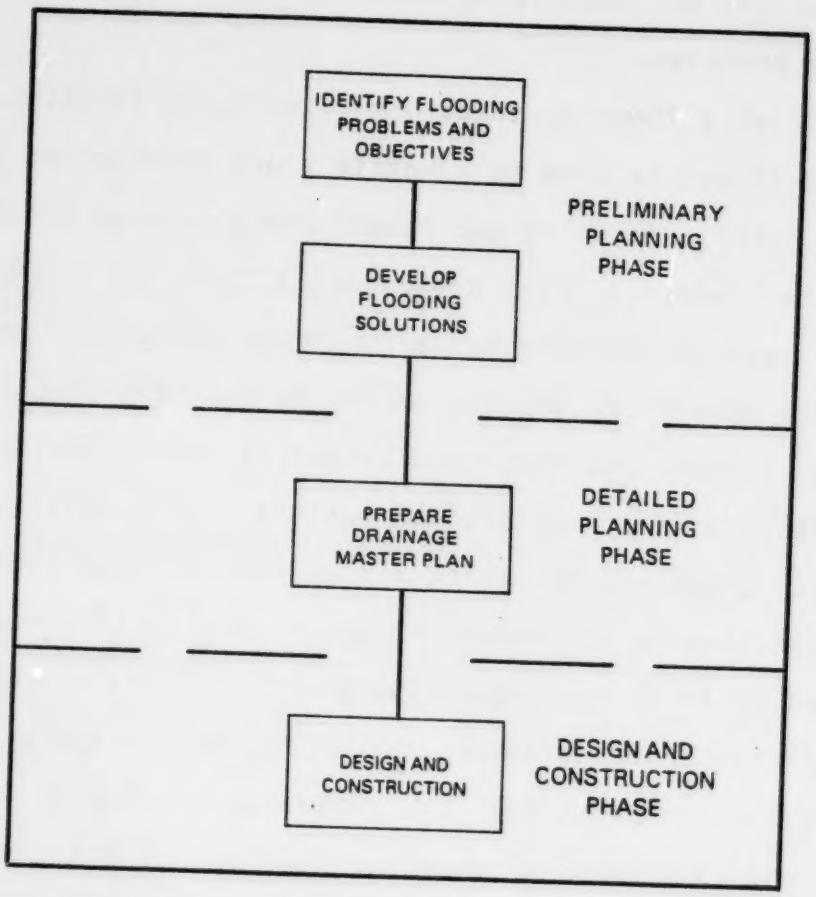


FIG. 2-13 STORMWATER QUANTITY MANAGEMENT PROCEDURE

through 2-5 can be used to identify the needed storage or capacity increases or runoff reduction to mitigate flooding. These storage and capacities would be required to solve the flooding problems.

Develop Flooding Solutions - The flood solution curves of Figure 2-11 can be used to identify every combination of storage volume, outflow capacity and runoff for a subarea which would alleviate flooding. Data presented in Table 2-7 can then be used to calculate preliminary costs for these measures. Once the available subset of flooding solutions has been identified for a subarea, optimal solutions can be sought which consist of some combination of flood mitigation options. This must be based on costs and practicality of the implementation practices. General costs can then be estimated. However practicality must be assessed based on working knowledge of the subarea. For example, land available for detention basins may be limited in a congested subarea such as Tampa Heights, excluding altogether storage volume enlargement as a practical option. At the end of this phase, the City will have a knowledge of the extent of the problem, the range of potential solutions and their costs, and a preliminary direction for the detailed planning and evaluation phase.

Detailed Planning Phase. It is the intent of this Phase of stormwater quantity management planning to integrate the information developed in the Tampa NURP Study with the existing City Master Drainage Planning (MDP) effort.

The detailed planning phase and MDP utilize assessments based on more detailed analyses such as the SWMM. The existing MDP scope of work is shown in Table 2-10. Only slight modifications to this scope would be necessary. Tasks 3 and 5 have been performed to some extent during this study. In addition, the information generated during the preliminary planning phase can serve as input to the detailed analysis. The

TABLE 2-10. SCOPE OF MASTER DRAINAGE STUDY

Task No.	Existing task title	Revised task title
1	Determination of Drainage Basin Boundaries	Review of Preliminary Planning Information
2	Inventory of Existing Drainage Structures	Determination of Drainage Basin Boundaries
3	Identification of Flood-Prone Areas	Inventory of Existing Drainage Structures
4	Collection of Hydrologic Data	Identification of Flood-Prone Areas
5	Development of Design Storm Event	Collection of Hydrologic Data
6	Development and Verification of Stormwater Model	(1)
7	Development and Evaluation of Alternative Programs	(1)
8	Selection of Optimum Program	(1)
9	Implementation Plan	(1)
10	Preparation of Final Report	(1)

1. Same Task Title for Existing and Revised Scope.

revised scope of work is also shown in Table 2-10. The revised MDP scope as it relates to the NURP study is described below.

Task 1. Review of Preliminary Planning Information - To perform the MDP, the flooding objectives and potential solutions must be reviewed as a starting point for the detailed planning phase.

Task 2 through 5. Collection of Drainage System Data - These tasks involve the collection and review of detailed site data. This includes catchment characteristics such as land use, impervious area, and existing stormwater controls; and drainage system characteristics such as pipe sizes, slopes, inverts and shapes. These data must be compiled and mapped and prepared as input to the SWMM for detailed analysis of the storm sewer system. Other information such as availability of land for retention ponds, desirability of ditches, etc. must also be collected. In some cases, some field data on stormwater flow may also be needed. Such data may be required for SWMM verification.

Task 6 through 10. Plan Evaluation and Selection - These tasks include analysis of the drainage area using SWMM, development and analysis of alternative solutions, and selection of a recommended plan. It should be noted that the SWMM calibration performed during the Tampa NURP study can be used as a starting point in the analysis of the basin. Also, information generated in the preliminary planning phase (i.e. needed flow reductions and potential control alternatives and their costs) will be most helpful in this analysis. Upon completion of these

tasks, the project can advance into the design and construction phase.

Summary. The framework presented here for stormwater quantity management has integrated current City practices with the results of the Tampa NURP study. The issues of stormwater quality and stormwater permitting will be dealt with in the next Chapters by expanding the framework laid out in this Chapter. The applications of the techniques presented in this Chapter will be demonstrated in Chapter 5, using a portion of the Sunshine Park subarea as a case study.

REFERENCES

1. American Society of Civil Engineers, Manuals and Reports on Engineering Practice, No. 37, Design and Construction of Sanitary and Storm Sewers, 1969.

2. City of Tampa, Florida, Drainage Design Criteria, January, 1977.

3. Metcalf & Eddy, Task II.2 Rainfall Quantity Analysis, City of Tampa, Tampa Nationwide Urban Runoff Program Phase II, February, 1983.

4. U.S. Department of Commerce, Rainfall Frequency Atlas of the United States, Technical Paper No. 40, May, 1961.

5. State of Florida, Florida Department of Transportation Drainage Manual.

6. City of Tampa, City of Tampa Preliminary Subsector Plans, Tampa 2000 Land Use Element, 1980.

7. Metcalf & Eddy, Phase II Report, Task II.6 Runoff Characterization - Data Analysis, Task II.9 Control Testing - Data Analysis, Task II.10 Projection of Study Area-Wide Loadings, City of Tampa, Tampa Nationwide Urban Runoff Program Study, October, 1983.

CHAPTER 3

STORMWATER QUALITY MANAGEMENT

General

Effective stormwater management planning in the City of Tampa must take into account the quality of stormwater and its effect on the lower Hillsborough River. The quantity (flooding) analysis in Chapter 2 will therefore be extended in this Chapter to take into account quality considerations.

Stormwater quality management objectives are mainly a function of the needed reductions of stormwater pollutant load to the lower Hillsborough River from the drainage area. The following information is therefore presented:

1. Identification of potential problem pollutants in stormwater with respect to water quality standards.
2. Selection of a design storm which will have the worst impact on the receiving water quality.
3. Estimation of design storm loads for potential problem pollutants.
4. Analysis of the impact of design storm loads for potential problem pollutants on the receiving water.
5. Determination of needed stormwater load reduction in the lower Hillsborough River.
6. Development of a stormwater quality management approach.

The information presented in this Chapter is integrated with the procedures presented in Chapter 2 for flooding, so that the quality aspects of stormwater management can be taken into account in preliminary and detailed planning efforts.

Potential Problem Pollutants

Table 3-1 compares the lower Hillsborough River water quality criteria with the quality of stormwater and of the River. Pollutants which exist in stormwater at concentrations which may violate the River criteria are indicated in the table. These include BOD (through its effect on DO), suspended solids, total phosphorus, total and fecal coliform, copper, lead, iron and mercury.

Upon their entry into the River, these pollutants will be diluted because of mixing with River water. This dilution must be taken into account when comparing the stormwater data with concentrations in the River. The parameters copper, lead and iron will need less than 3 dilutions to meet River criteria. Since this dilution will generally be attained in the immediate vicinity of an outfall, these parameters are not expected to violate criteria. However, it is conceivable that deposition of these metals to the bottom sediments and subsequent resuspension to the River may occur. This question is addressed in the City of Tampa's Receiving Water Study[1]. Other parameters such as mercury and coliform bacteria require much larger dilutions to satisfy the River criteria, and therefore are expected to be in violation at some locations in the River.

TABLE 3-1. COMPARISON OF LOWER HILLSBOROUGH RIVER WATER QUALITY CRITERIA WITH STORMWATER AND RIVER DATA

Parameter	Unit	Criteria	Mean Conc. of Pooled Runoff Data	River Data
BOD_5	mg/l	shall not cause DO depressions violating standards	14(1)	1-20
COD	mg/l	-	51	7-106
TSS	mg/l	settle to form putrescent deposits or a nuisance; float as debris or scum; produce a turbidity nuisance; <29 NTU's above background; depth of compensation <10% change.	40(1)	1-71
Nutrients:				
Ortho P	mg/l	man-induced nutrient load shall not cause imbalances in natural populations	0.14	0.08-0.93
Total P	mg/l		1.38(1)	0.15-0.95
Nitrate	mg/l		1.89	0.001-0.940
Nitrite	mg/l		0.02	0.000-0.022
TKN	mg/l		1.24	0.15-2.15
Ammonia-N	mg/l	0.02	0.38	0.044-0.77
Organic-N	mg/l		0.99	0.38-5.6
Total-N	mg/l		1.66	0.71-6.1
Fecal Coliform	MPN / 100 ml	monthly avg = 200 10% >400 daily avg. = 800	24,800(1)	40-26,500
Total Coliform	MPN / 100 ml	monthly avg. = 1,000 20% >1,000 never >2,400	156,200(1)	300-600,00

(1) Potential Problem Pollutant

METCALF & EDDY

TABLE 3-1 (Continued). COMPARISON OF LOWER HILLSBOROUGH RIVER WATER QUALITY CRITERIA WITH STORMWATER AND RIVER DATA

Parameter	Unit	Criteria	Mean Conc. of Pooled Runoff Data	River Data
Aluminum	ug/l	1,500		50-530
Cadium	ug/l	5	2.49	-
Copper	ug/l	15	49(1)	3-57
Lead	ug/l	50	57(1)	4-26
Zinc(2)	ug/l	30	59	4-90
Arsenic	ug/l	50	9	-
Barium	ug/l	-	35	-
Chromium	ug/l	50	8.4	-
Iron	ug/l	300	545(1)	-
Manganese	ug/l	-	14	-
Mercury	ug/l	0.1	6.9(1)	-
Nickel	ug/l	100	13	-

1. Potential Problem Pollutant
2. Freshwater criteria = 30, general criteria = 1,000

For some pollutants discharged to the River by stormwater, the background concentrations in the River (due to other sources as well as stormwater) already violate the criteria. For these parameters, it must be determined whether reduction of stormwater loads alone will achieve the criteria. A discussion of the potential effect of stormwater quality parameters on the lower River follows. This discussion is a summary of information

presented in the City of Tampa's Receiving Water Study Report[1]. Later in this Chapter, the effect of these parameters on the lower River is quantified, where applicable.

Coliform Bacteria. The most clearly identifiable effect of stormwater on the lower Hillsborough River is the impact of coliform bacteria. A large increase of bacteria occurs during and after a storm event. Violations of the criteria by 1 or 2 orders of magnitude, or even higher, occur. Therefore, extensive reduction of the coliform load would be required for River standards to be consistently met.

BOD and DO. The dissolved oxygen quality criteria state that the concentration shall not average less than 5 mg/l over a day and shall never be less than 4 mg/l. Existing DO levels that are attributable to other conditions such as intrusion of Bay water often violate these criteria. The DO sag produced by stormwater BOD should be investigated in order to determine the contribution of stormwater in relation to other effects.

Total Suspended Solids. The total suspended solids (TSS) quality criteria are that TSS concentrations will not:

1. Reduce the depth of the compensation point for photosynthetic activity by more than ten percent below the natural background level.
2. Settle to form putrescent deposits.
3. Float as debris, scum, oil or other matter in such amounts as to form nuisances.
4. Increase turbidity more than 29 NTU above background.

The concentration of solids in stormwater from Tampa is relatively low compared to national data. Some increases in River solids during storm event measurement programs do occur. However, the solids input is not expected to violate the depth of compensation criterion for photosynthetic activity or the numerical turbidity criterion. A potential impact of solids, however, is the associated particulate metal content which they may carry into the River. The criterion regarding floatables is difficult to quantify. There are undoubtedly some negative impacts due to washout and discharge of trash, oils, etc. The only method to effectively remove such matter would be collection and treatment. The large number of existing outfalls that would require treatment, and the associated large expense, precludes this. Two methods that would have beneficial impact are installation of bar racks at major outfalls and a good municipal litter control program instituted with a public education program.

Metals. As mentioned, the mercury input from stormwater is significant when compared with the water quality criterion while copper, lead and iron are borderline. Recycling of these metals from the sediment to the water column may occur. Some reduction in the metals content of stormwater may prove beneficial.

Total Phosphorus. The mean concentration of phosphorus in the pooled runoff data is only slightly higher than the River data. Therefore, average induced total phosphorus enrichment

would be minor. In addition, the input of phosphorus from the upstream drainage basin and the Bay make it unlikely that control of phosphorus in stormwater in the lower River will result in any impact on the River.

The effect of these parameters on the lower River during a design storm event will be discussed in more detail later.

Selection of Design Storm

The adverse impact of a storm event on the receiving waters is associated with the washoff of surface solids caused by the storm. For a series of design storms, the shortest duration, lowest intensity storm that will still wash off most of the surface solids will have the most adverse impact on the receiving waters because it will have both maximum loads and concentrations.

A series of design storms with return periods of one, three, six and twelve months and durations of one, two, three and six hours was developed to find this worst case storm. The development of these storms required performing an intensity-frequency-duration analysis on the available rainfall record and then constructing hyetographs from the resulting rainfall curves. This development was described in detail in the output for Task II.2 Rainfall Analysis[2] of the Tampa NURP Project. Figure 3-1 illustrates the intensity-frequency-duration curves for Tampa.

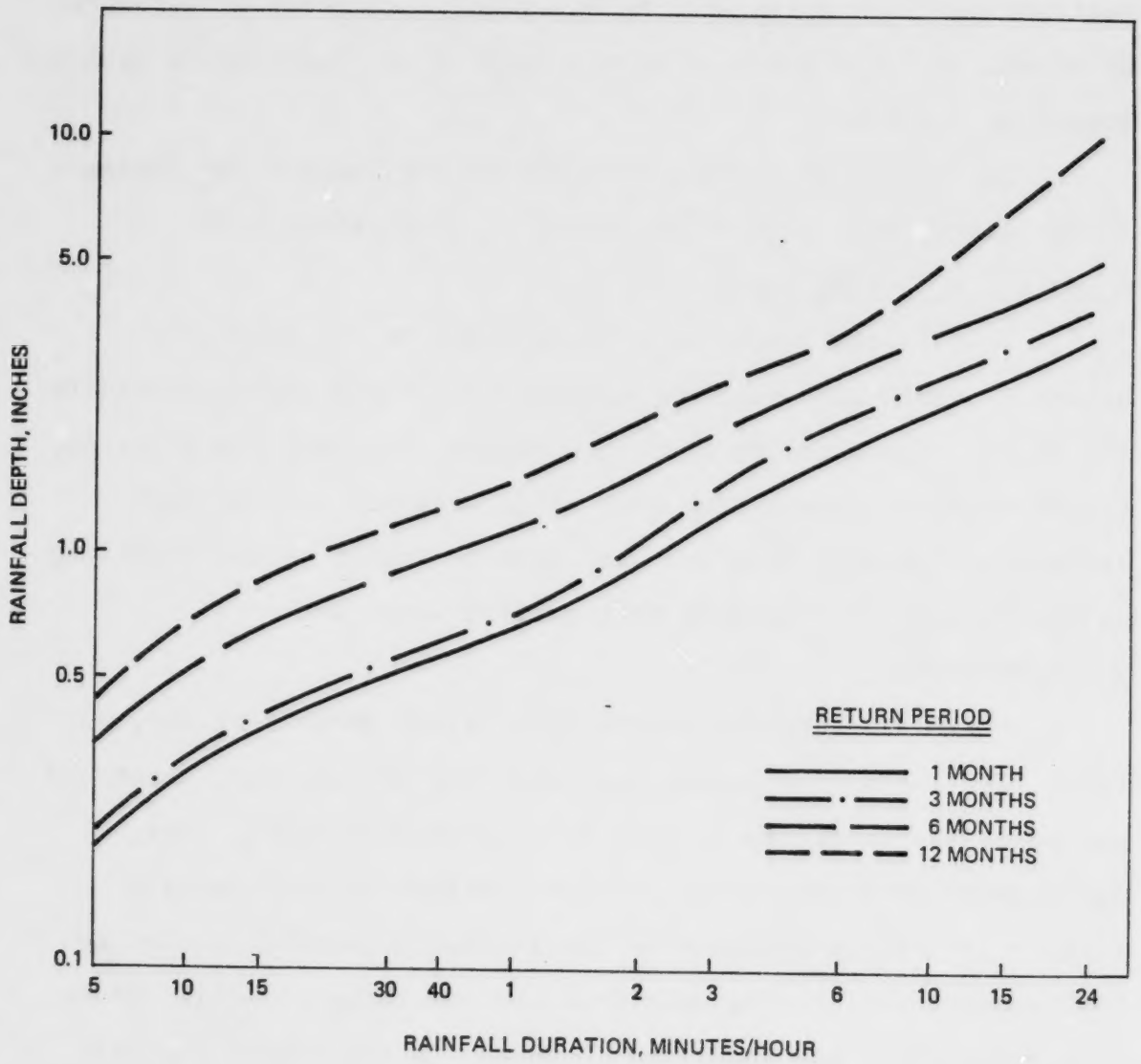


FIG. 3-1 RAINFALL INTENSITY-FREQUENCY-DURATION CURVES
FROM TAMPA INTERNATIONAL AIRPORT RAINFALL RECORDS

A rainfall hyetograph is a plot of time in hours versus intensity in inches per hour. The area under a hyetograph histogram is the total rainfall in inches. Figure 3-2 shows the one-month two-hour rainfall hyetograph with a peak 20 minute intensity of 1.29 inches per hour and a total rainfall of 0.96 inches.

In Phase II of this study, the quantity and quality response of five subbasins predicted by Metcalf & Eddy's version of the Storm Water Management Model (SWMM) (3) was calibrated against an extensive set of field data (4). The quality parameters determined during the calibration of the subbasins are listed in Table 3-2.

The physical characteristics of the previously calibrated subbasins are shown in Table 3-3. These subbasins are composed primarily of low density residential and commercial/industrial lands as in the lower Hillsborough River basin. The locations of the subbasins are shown in Figure 3-3. Pollutant concentrations that are the average of the composite, flow weighted concentrations from the five subbasins are expected to be fairly representative of average concentrations from the overall study area.

In a basin with a low solids accumulation rate, it is conceivable that all sixteen design storms would wash off all of the surface solids. In a basin with a high accumulation rate, this is clearly less likely to happen. Therefore, in order to

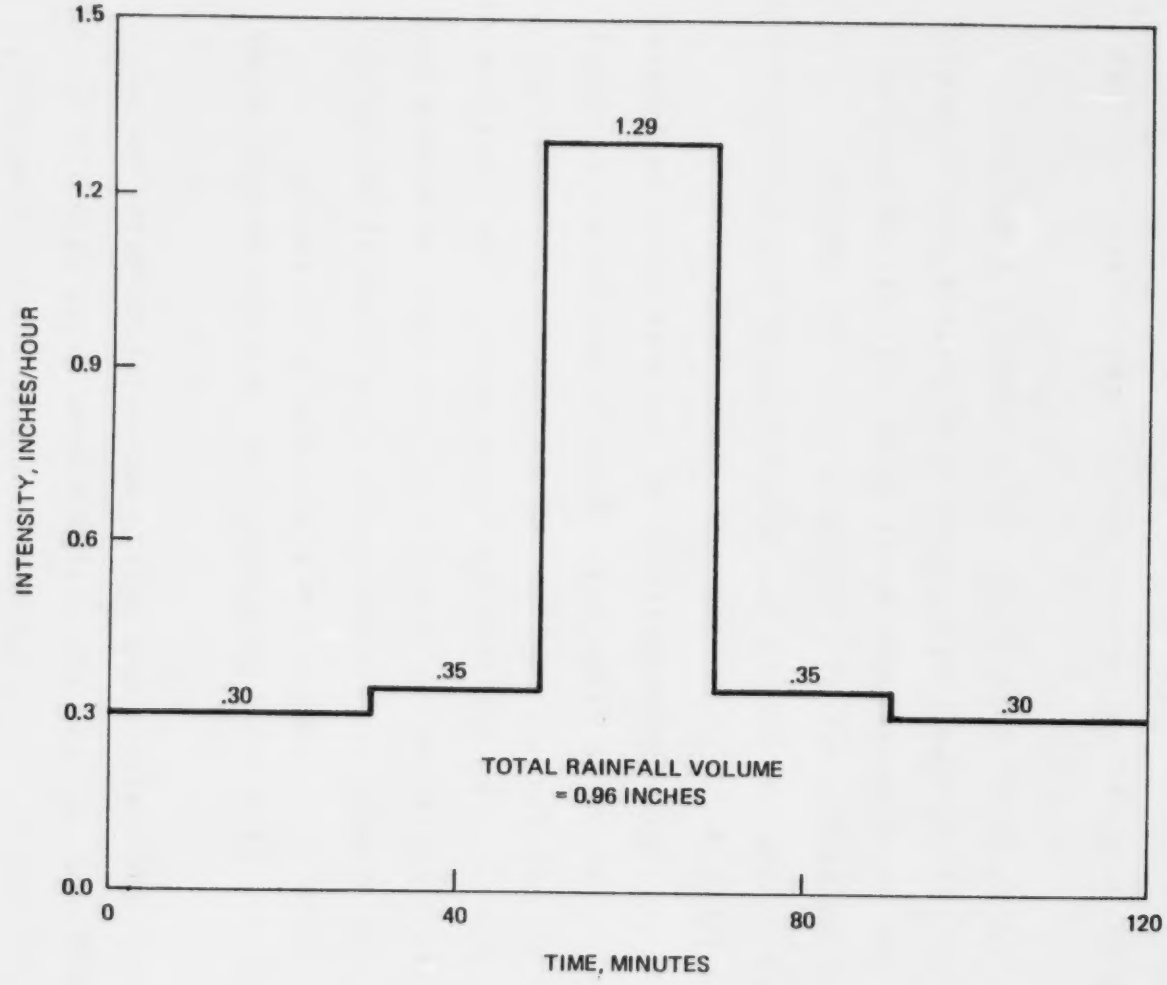


FIG. 3-2 RAINFALL HYETOGRAPH FOR THE ONE MONTH, TWO HOUR DESIGN STORM

METCALF & EDDY

TABLE 3-2. STORMWATER QUALITY PARAMETERS OF SUBBASINS ESTABLISHED DURING CALIBRATION OF SWMM

Basin	Land use(1)	Percent land use in basin	Dust and dirt per 100 ft. gutter, (lbs.)	Soluble BOD per lb dust and dirt (mg/gm)	Non-soluble BOD(2)	Coliform multiplier(3)
J.L. Young	2	100	0.05	20	0.2	5.0
Wilder Ditch	1	51	0.04	50	0.6	1.0
	2	24	0.18	77		
	3	7	0.26	30		
	4	17	0.08	50		
North Jesuit	1	58	0.5	25	0.25	1.0
	4	42	2.2	38		
Charter & Harding	1	90	0.04	10	0.10	5.0
	4	10	0.08	10		
Norma Park	1	7	0.01	5.0	0.05	5.0
	3	93	0.02	7.7		
SWMM Default Values	1	-	0.7	5.0	0.05	1.0
	2	-	2.3	3.6		
	3	-	3.3	7.7		
	4	-	1.5	5.0		

1. Low density residential = 1, high density residential = 2, commercial/institutional = 3, open space/natural = 4.
2. Fraction of dust and dirt that is non-soluble BOD.
3. Multiplier used for coliform calibration.

TABLE 3-3. PHYSICAL CHARACTERISTICS OF THE CALIBRATED SUBBASINS

	Size, Acres	Percent Impervious	Percentage Area for Given Land Use Type				
			Industrial/ Commercial	High Density residential	Medium Density residential	Low density residential	Institutional/ open space
J.L. Young	8.8	61		100			
Wilder Ditch	193.9	39	25			55	21
North Jesuit	29.5	37				48	52
Charter & Harding	42.2	31				89	11
Norma Park	46.6	90	91		9		

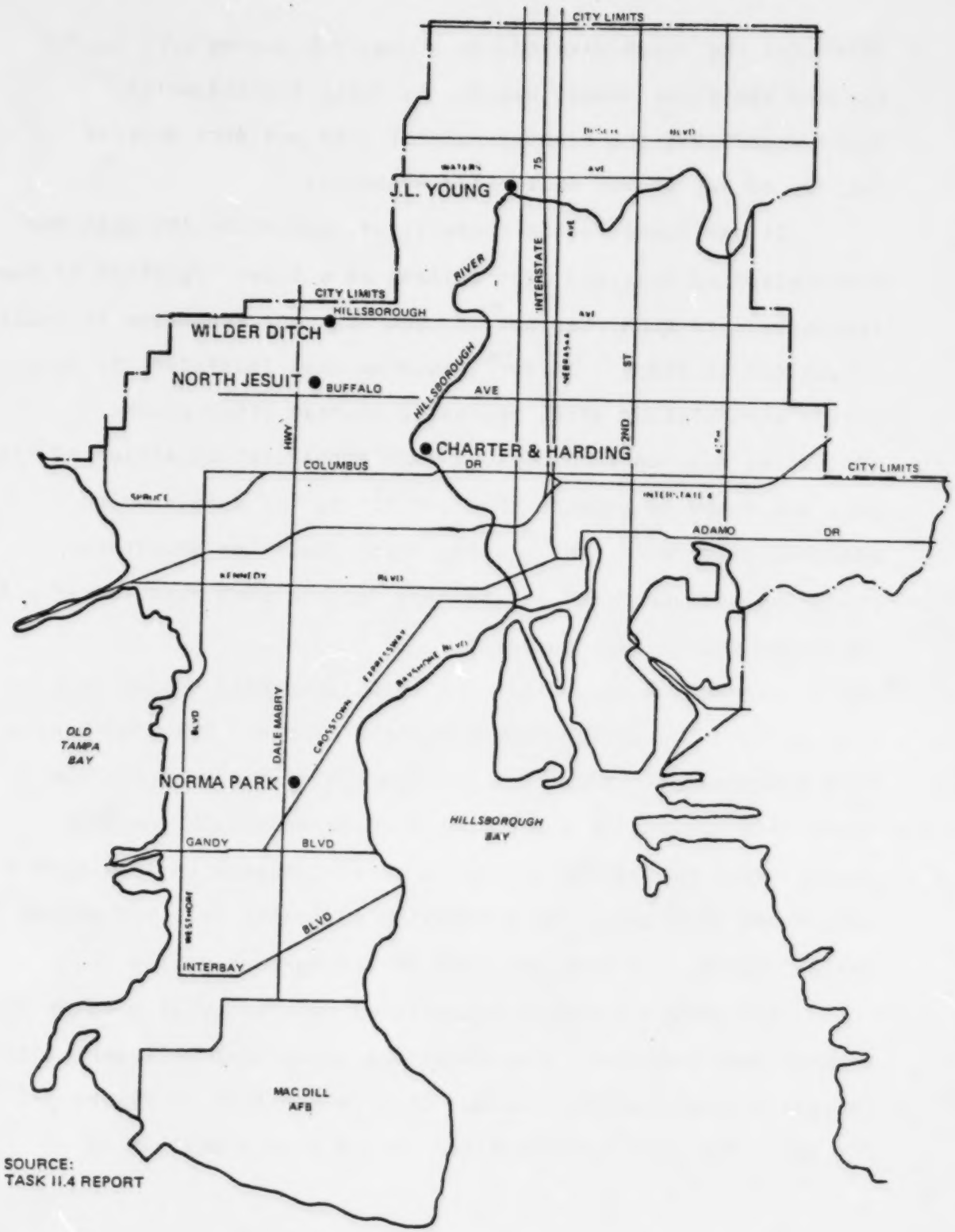


FIG. 3-3 SITE LOCATIONS OF THE CALIBRATED SUBBASINS

determine the worst case design storm, the storms were routed through the North Jesuit basin. As Table 3-2 indicates, North Jesuit has the highest rate of dust and dirt surface buildup of any of the calibrated subbasins.

It was found during Phase II of this study (4) that the computation of dust and dirt buildup as a linear function of time (assumption in SWMM) was not an accurate representation of actual conditions in Tampa. In fact, studies have indicated (5) that solids accumulation often reaches a plateau after which additional buildup is minor. A SWMM antecedent condition of six days was found to provide the best fit to the events which occurred after more than six dry days. Thus the antecedent condition was specified as six days for the SWMM runs to reflect the maximum pollutant build-up.

Figure 3-4 shows that the total suspended solids (TSS) load between the design storms changes little. TSS load varies from a maximum of 581 pounds for the twelve-month three-hour storm to a minimum of 522 pounds for the one-month one-hour storm. This represents only a 10 percent change in load over the entire set of storms. This behavior indicates that all of the design storms will wash off most of the surface solids.

The design storm of least total rainfall will produce the highest mean pollutant concentrations given that pollutant load between storms changes little. This is verified in Figure 3-5 for TSS. TSS mean concentration varies from a maximum of

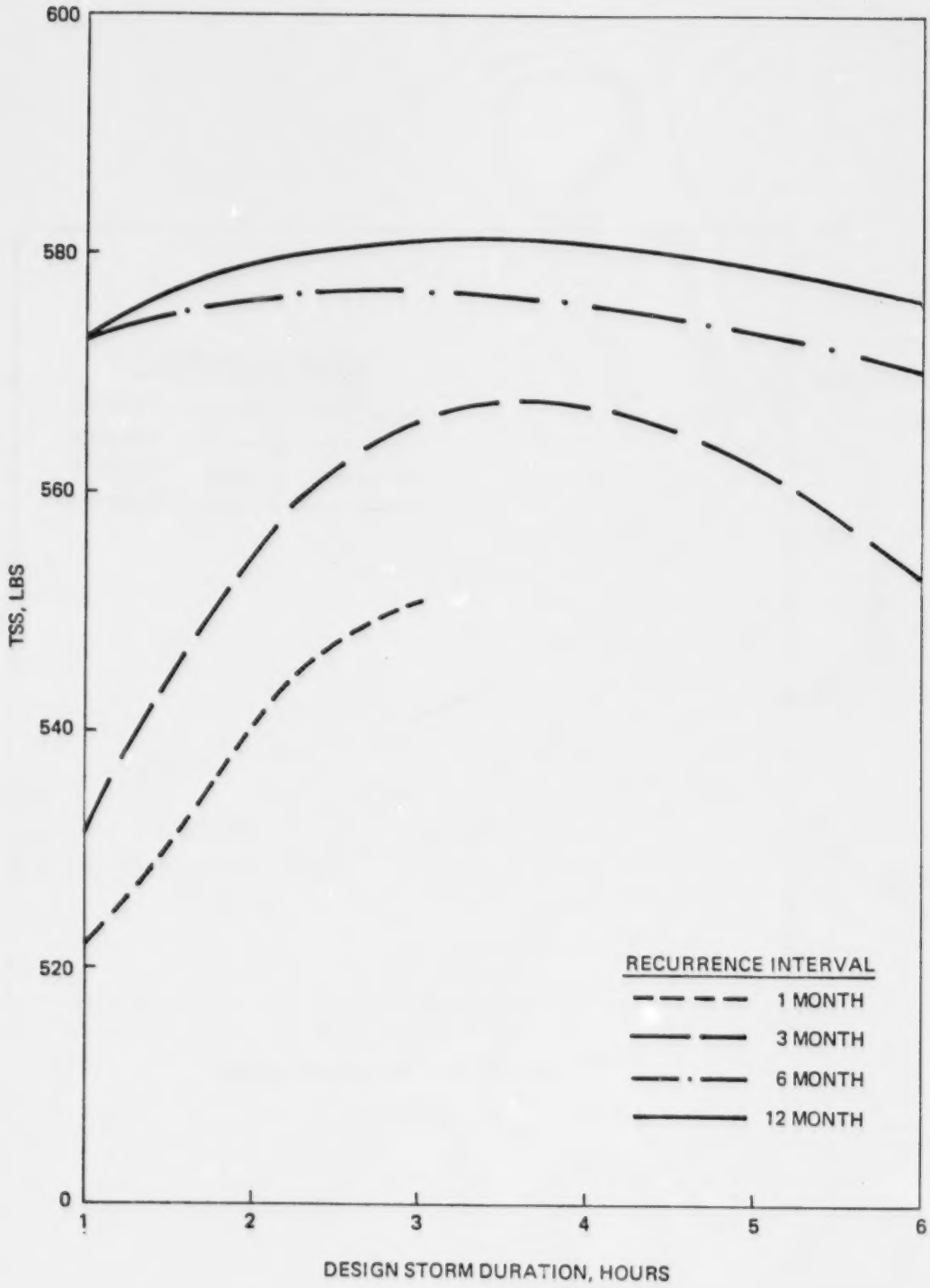


FIG. 3-4 TOTAL SUSPENDED SOLIDS LOAD
FOR THE DESIGN STORMS IN THE NORTH JESUIT BASIN

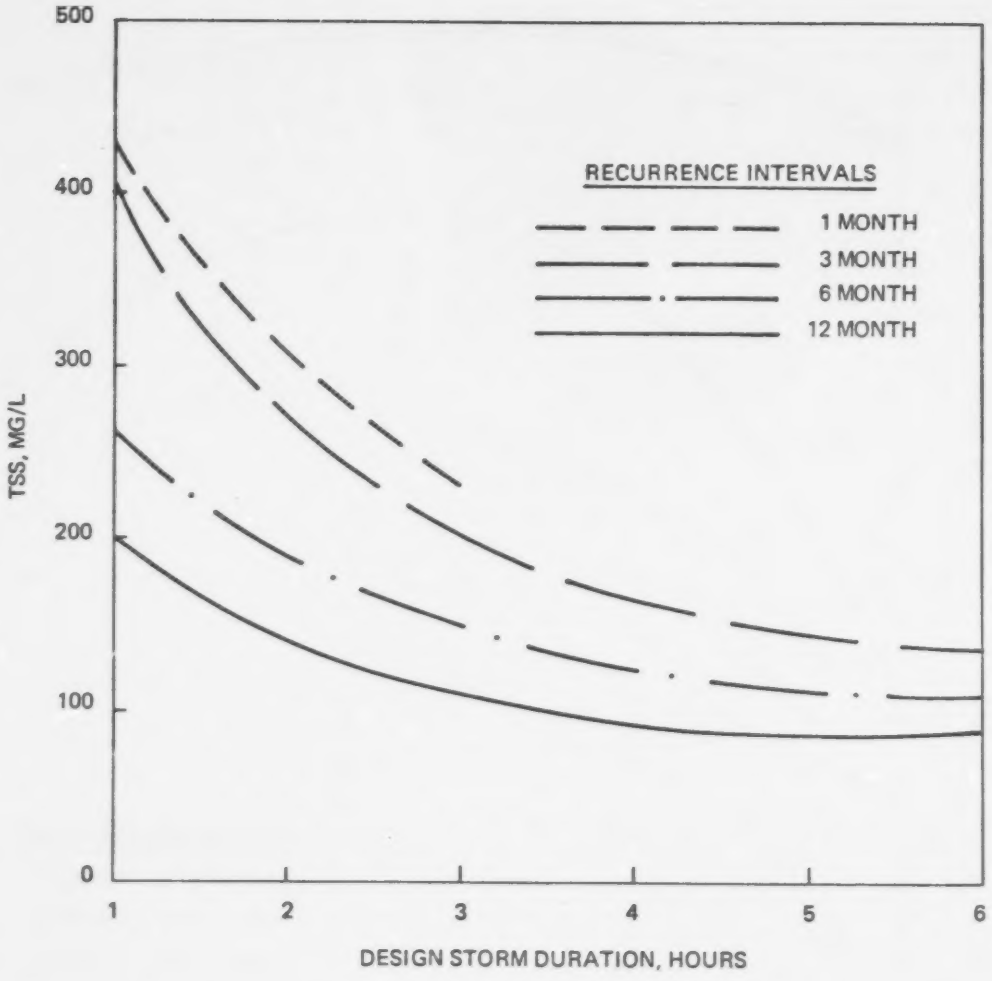


FIG. 3-5 MEAN TOTAL SUSPENDED SOLIDS CONCENTRATION FOR THE DESIGN STORMS IN THE NORTH JESUIT BASIN

429 mg/l for the one-month one-hour storm to a minimum of 90 mg/l for the twelve-month six-hour storm.

Concentrations of BOD_5 and coliforms are determined in SWMM as a function of TSS. As a result, they will behave similarly to TSS. Thus, it is concluded that the one-month one-hour design storm has the worst impact on the River. This storm has a total rainfall of 0.66 inches.

However, design storm consistency with State stormwater permitting requirements must also be considered in the design event selection. The Florida Department of Regulation (DER) Stormwater Rule requires retaining the runoff from the first inch of rainfall. It is important to demonstrate the effect of area-wide implementation of this requirement for the stormwater permit. Several of the design storms have approximately one inch of total rainfall. Of these, the one-month two-hour storm has the worst impacts on the River. This storm is second to only the one-month one-hour storm in terms of River impact. For these reasons, it is decided to continue this analysis with the one-month two-hour event.

Design Storm Pollutant Loads

Having established the one-month return period, two-hour duration storm as the water quality design event, this storm was then routed through the four remaining subbasins with the SWMM in order to determine pollutant concentrations representative of the entire lower Hillsborough River basin. This storm was then routed through the overall study area with the representative

concentrations using the Simplified Stormwater Management Model (S/SWMM) (6). A description of the representative pollutant concentration simulations and the S/SWMM results follows.

Mean, flow weighted concentrations of TSS, BOD_5 and fecal coliforms were calculated for all five subbasins. The five values for each pollutant were averaged to give the following representative concentrations associated with the design event:

TSS = 77 mg/l

BOD_5 = 21 mg/l

Fecal Coliforms = 31,100 MPN/100 ml

The above concentrations are slightly different from the means of the composite data collected in Phase II. This is because they are representative of one specific storm, rather than an average of many storms. These concentrations are used in simulations with the S/SWMM.

Study Area-wide Design Storm Loads. The design storm with the representative pollutant concentrations was routed through the entire study area with the S/SWMM. The hyetograph of this design storm was shown in Figure 3-2. Table 3-4 summarizes the results by subarea in order of decreasing runoff volume and pollutant load. Although the storm lasts only two hours, runoff occurs for up to eleven hours after the storm, thus attenuating the impact on the river. None of the subareas flood during this event.

TABLE 3-4. DESIGN STORM POLLUTANT LOADS BY SUBAREA

Subarea Abbrev.	Runoff Volume, mgal	Hours of runoff	TSS, lbs.	BOD_5, lbs.	Fecal Coliforms MPNx10^{12}
FH	20.7	4	13,302	3,636	24
SJ	12.8	2	8,234	2,250	15
TH	12.6	3	8,090	2,211	15
SC	12.3	6	7,895	2,158	14
HA	12.2	2	7,826	2,139	14
KC	12.1	2	7,783	2,127	14
NW	11.8	11	7,594	2,075	14
NT	11.1	11	7,173	1,960	13
UT	10.1	2	6,506	1,778	12
WT	10.1	2	6,481	1,771	12
RB	9.8	2	6,301	1,722	12
SP	9.6	2	6,178	1,689	11
DT	8.6	2	5,552	1,517	10
WW	8.5	2	5,478	1,497	10
SH	6.6	2	4,247	1,161	8
SS	6.2	2	3,960	1,082	7
OG	4.7	2	2,991	818	5
Total	179.8		115,591	31,591	210

The maximum load during the peak rainfall intensity hour, which corresponds to the critical pollutant shock, comes from the St. Joseph's subarea followed by Kirby Creek and the University of Tampa. These subareas contribute runoff for 2 hours and thus have higher pollutant concentrations associated with them than the subarea with the highest overall load, Forest Hills, which takes 4 hours to runoff. The time variation of cumulative TSS load to the lower River from all seventeen subareas is illustrated in Figure 3-6. Runoff, BOD_5 and fecal coliform bacteria behave similarly. It is this time varying behavior which is input to the lower Hillsborough River model.

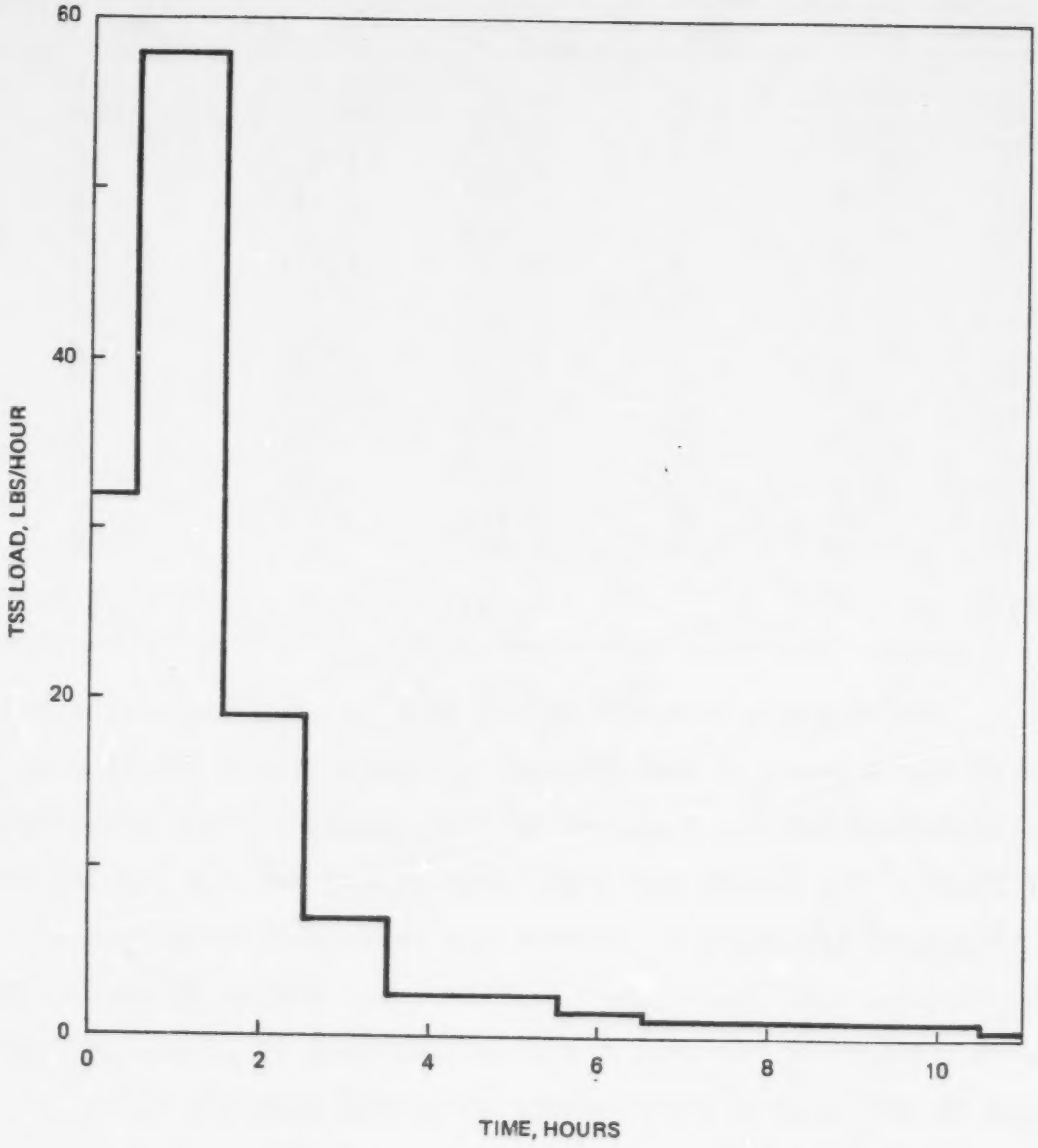


FIG. 3-6 TSS DESIGN STORM RUNOFF, LOAD HISTOGRAM
TO LOWER HILLSBOROUGH RIVER FROM OVERALL STUDY AREA

Receiving Water Impacts

Having quantified the worst case stormwater flows and loads to the lower Hillsborough River, their impact on the River is analyzed with the Water Quality in Open Channel Networks (WQON) model (7). The WQON modeling effort was performed in a related study (1) and is summarized here.

In the WQON model, well channelized estuaries such as the lower Hillsborough River are modeled as a network of one-dimensional reaches. Hydrodynamics, conservative substances and non-conservative substances which can be approximated by a first order decay rate can be modeled under time varying, dynamic conditions. The interaction between BOD and DO can be modeled. Lateral inflows with associated pollutant concentrations can be specified in the model along the reaches, allowing simulation of stormwater runoff. In this study, the model is used in a predictive capacity to examine the effect on the lower Hillsborough River of the worst case stormwater inflow condition.

For analysis of stormwater impacts, the parameters coliform, BOD/DO, solids and metals are mainly of interest. Modeling of coliform and BOD/DO was performed. The complex interactions of solids and metals between particulate and dissolved phases of the water column and sediments was not modeled. A more qualitative evaluation of these parameters is given below.

A dry weather, low flow, river condition with no freshwater release from the dam was modeled with stormwater

inflow commencing at low water slack. A stormwater lateral inflow was associated with each individual subarea. Low water slack was selected so that the pollutant flushing effect of the ebb tide was postponed as long as possible. The impact of the stormwater pollutant load will be most critical for these River hydraulics. The model input values are summarized in Table 3-5. The basis of these model inputs is described below.

Background and Initial Conditions - Initial water surface elevations at low water slack were ascertained from tide gage recordings at five locations along the River. The background profiles of BOD and DO concentration were the same as those recorded during a synoptic survey conducted in April, 1982. These background conditions are representative of an extended period of dry weather, which fosters low DO levels. During dry weather, freshwater releases from the Reservoir will be small, the River will not flush its pollutant load and DO will be depleted. The background coliform concentrations were taken from the February, 1983 synoptic survey. The DO saturation concentration of 7.3 mg/l is based on a River temperature of 21°C, and an average river salinity of 15 ppt.

Boundary Conditions - Downstream water surface variation was ascertained from tide gage recordings at Platt St., which is near the mouth of the River. The freshwater flow being released from the lower Hillsborough River Dam, which is the upstream boundary, is specified as zero. The inflow from Sulphur Springs is assumed to be 10 cfs.

TABLE 3-5. SUMMARY OF WQON MODEL INPUTS FOR DESIGN STORM IMPACT ASSESSMENT

Background and Initial Conditions		
Hydrodynamic - water surface elevation		Feb. 1, 1983 data
Water Quality - background quality		April, 1982 synoptic survey
	- dissolved oxygen saturation	7.3 mg/l (corresponds to temperature of 21°C and salinity of 15 ppt)
Boundary Conditions		
Hydrodynamic	- downstream tidal variation	Feb. 1, 1983 data
	- Hillsborough River flow	0 cfs
	- Sulphur Springs flow	10 cfs
Water Quality	- upstream concentrations	BOD = 1.5 mg/l DO = 7.3 mg/l Fecal Coliform = 300 mpn/100 ml
	- downstream concentrations	BOD = 2.5 mg/l DO = 5.7 mg/l Fecal Coliform = 500 mpn/100 ml
Stormwater Inputs		
Hydraulic - design storm flows		Table 3-4
Water Quality - design storm concentrations		Table 3-4
	- stormwater DO concentration	5 mg/l
Reaction Rates		
Benthic Oxygen Demand		2.6-9.8 $gO_2/m^2/day$
Deoxygenation Rate		.23 day^{-1}
Reaeration Rate		.17 - .32 day^{-1}
Coliform Decay Rate		2.0 day^{-1}
Temperature (for reaction coefficient correction)		21°C

The incoming tide was assumed to have a dissolved oxygen deficit (DOD) of 1.6 mg/l, corresponding to water of temperature 21°C and salinity of 35 ppt. BOD and fecal coliform concentrations were 2.5 mg/l and 500 mpn/100 ml, respectively. Upstream quality was established from the April synoptic survey, and is as follows:

- DO = 7.3 mg/l,
- BOD = 1.5 mg/l, and
- fecal coliform = 300 mpn/100 ml.

Stormwater Inputs - Quantity and quality are determined from the flows and loads predicted by S/SWMM which are detailed by subarea in Table 3-4. The total flows and loads were distributed over time as indicated in Figure 3-6. This was achieved through variable lateral inflows. Dissolved oxygen concentrations in stormwater measured during Tampa NURP Phase II (8) ranged from 1.0 to 7.3 mg/l with a mean value of about 5.0 mg/l. A survey of nine urban watersheds in the Tampa Bay Area performed by USGS (9) produced a range of 5.2 - 7.8 mg/l for stormwater DO concentrations with an average of 6.0 mg/l. A conservative DO concentration of 5.0 mg/l was selected for the design storm runoff.

Reaction Rates - The fecal coliform decay rate used is 2.0/day and the BOD decay rate used is 0.23 day^{-1}. Dissolved oxygen reaeration was calculated using the O'Connor-Dobbins formula and ranges from 0.17 to 0.32 day^{-1}. The benthic oxygen

demand rates ranged from 2.6 to 9.8 $gO_2/m^2/day$ as measured during a low flow period.

Simulation of Fecal Coliform. A profile of fecal coliform concentrations one tidal cycle after the storm is compared with initial conditions in Figure 3-7. The criteria for average daily fecal coliform count is also shown in this Figure. Clearly, violations of fecal coliform concentrations occur throughout most of the River for at least one day after a storm event.

Simulation of BOD/DO. Figures 3-8 and 3-9 show initial BOD and DO conditions in the River as well as concentration profiles one tidal cycle after the storm. As the BOD is exerted and flushed from the River in subsequent tides, the DO's may be reduced somewhat, but the general shape of the profile will remain the same. These Figures serve to demonstrate the effect of stormwater on DO and BOD concentrations in the River. The stormwater has little impact in reaches where initial DO levels are below 4 mg/l. In reaches with initial DO levels above 4 mg/l, the stormwater may cause a 1 to 3 mg/l drop in DO concentrations.

Analysis of available vertical DO profiles, which indicate low DO levels local to the riverbed, suggests that the DO violations during dry periods are due to the long-term exertion of benthic demand on an insufficiently flushed estuary. The benthic demand comes from organic deposition from the saline wedge intruding from the Bay. The longer River water resides

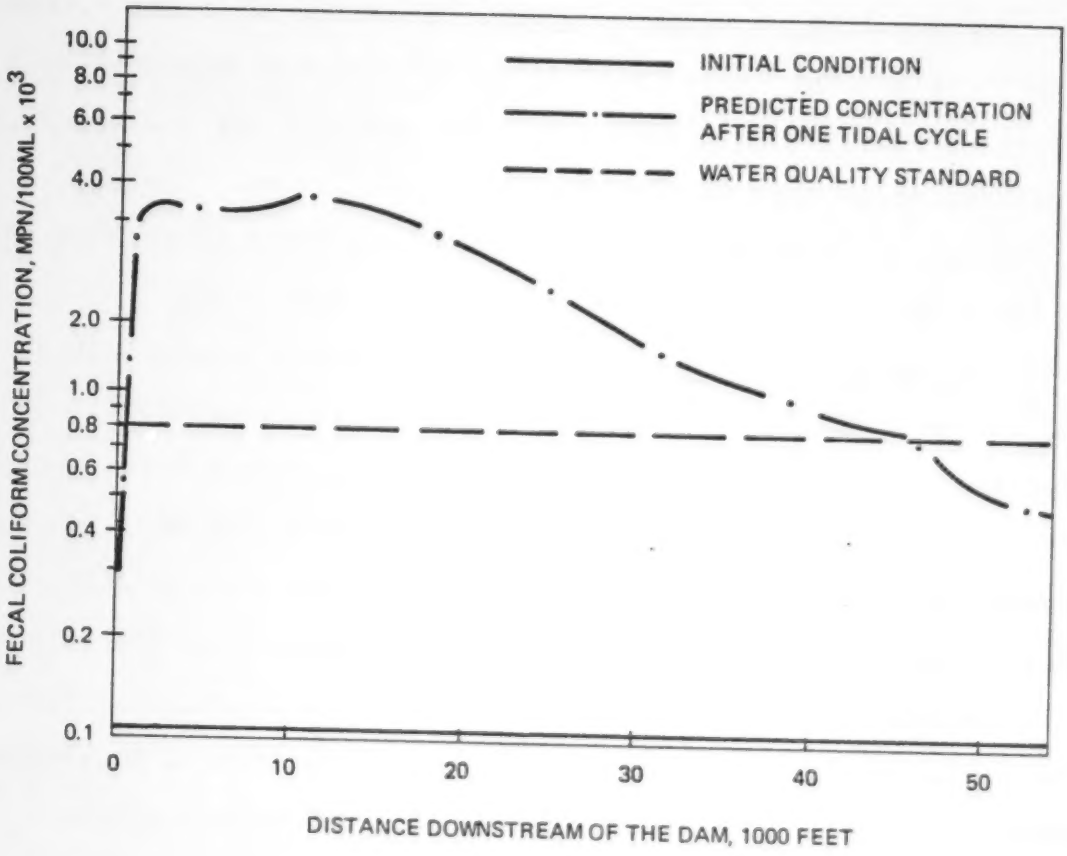


FIG. 3-7 INITIAL AND FINAL FECAL COLIFORM PROFILES ALONG RIVER FOR WQON SIMULATION

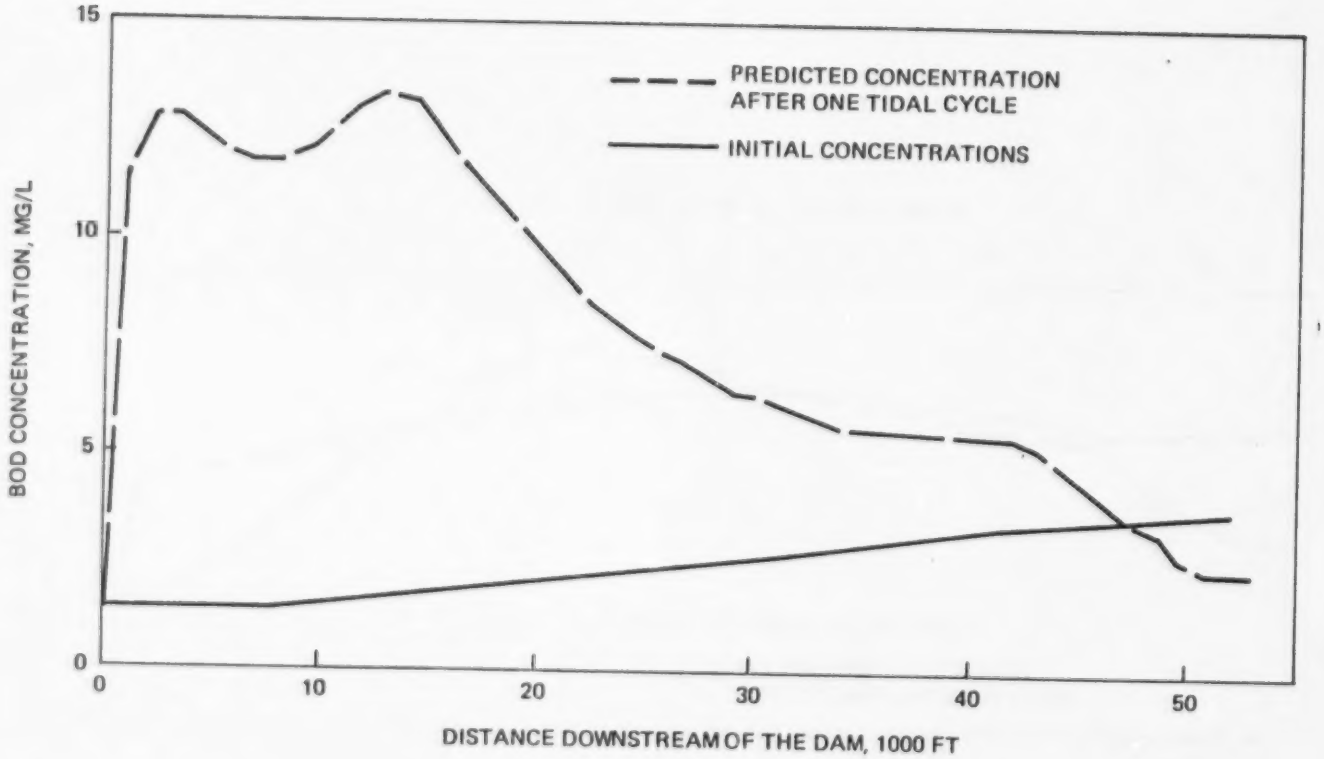


FIG. 3-8 INITIAL AND FINAL BOD PROFILES ALONG RIVER FOR WQON SIMULATION

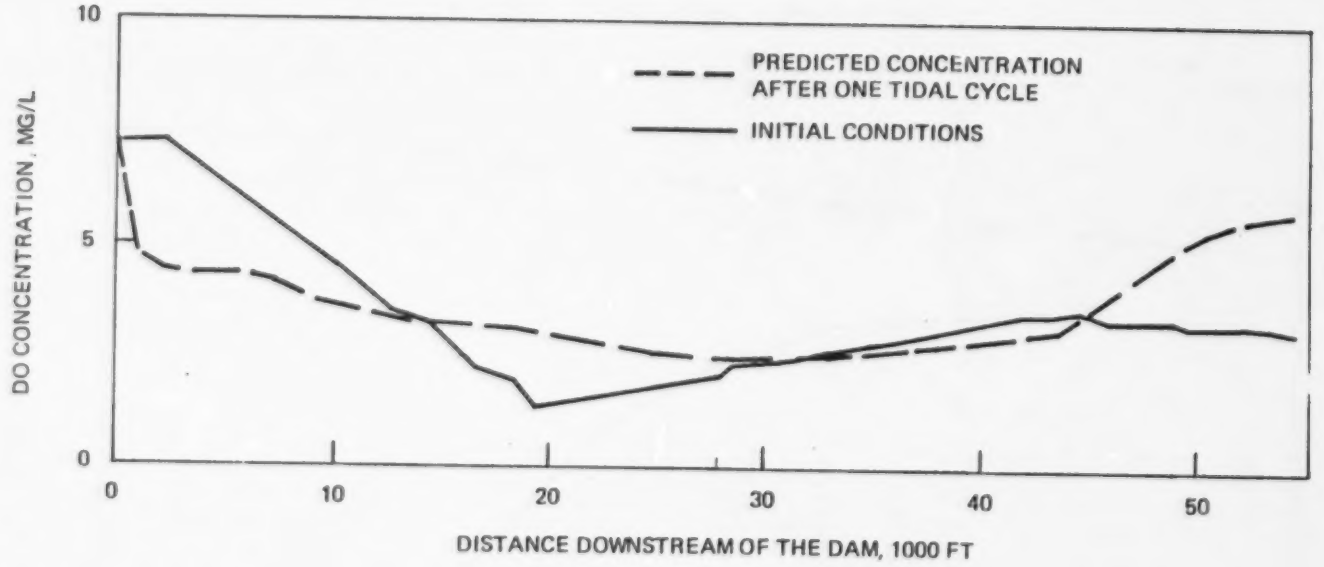


FIG. 3-9 INITIAL AND FINAL DO PROFILES ALONG RIVER FOR WQON SIMULATION

over these organic deposits, the more DO the benthos will be able to extract from it. Thus insufficient flushing due to low freshwater releases from the Dam leads to critical DO depletions. If benthic demand is removed, the River will return to nonviolation DO levels during dry periods. Additional simulations predict that the stormwater will cause a maximum DO sag of 2 mg/l when the river has initial DO levels of 5 mg/l or more. This may cause violation conditions depending on what the initial river profile will look like once benthic demand is removed.

Management of Stormwater Quality

The impact of stormwater loading on the lower Hillsborough River has been demonstrated. Based on the results presented in this Chapter, the following issues regarding stormwater quality are incorporated into the City of Tampa's SMP:

1. Receiving water objectives for stormwater
2. Ranking of problem areas
3. Ranking of stormwater load reduction techniques

These are explained in the following paragraphs.

Receiving Water Objectives. If nothing else was changed, it is unlikely that reasonable reductions in stormwater loads would result in attainment of all water quality criteria. For example, DO would not be in compliance if such reductions were implemented. This is due mainly to outside influences upon the quality of the lower Hillsborough River, including Bay water being transported upstream by the saline intrusion, and Reservoir

water being transported downstream. However, general strategies regarding desired load reductions in stormwater can be considered. These strategies are summarized in Table 3-6, and identified by parameter as follows:

TABLE 3-6. STORMWATER LOAD REDUCTION STRATEGIES

Issue	Parameters		
	Bacteria	BOD-DO	Solids[1] and Metals
Major Source	stormwater	Hillsborough Bay	stormwater, upstream drainage basin, and Bay
Correction Required to meet Water Quality Standard	collection and disinfection of over 99 percent of flow	removal of saline intrusion and benthic demand	some removal of metals from all sources[2]
Possible City of Tampa Strategies	remove or detain stormwater when possible	build dam, dredge bottom, maintain effective freshwater flow	remove or detain stormwater when possible

1. Solids do not violate standards, but metals do.
2. Required removal of mercury would be far greater.

1. Bacteria - Stormwater is a major source of bacteria in the lower Hillsborough River. However, in order to meet bacterial water quality criteria nearly all the stormwater flow would need collection and disinfection. Enactment of this is not feasible. Therefore, the City strategy could be to remove or

detain stormwater whenever possible. Removal by drain trenches or ditches will result in die-off of the bacteria. Detention will also result in significant coliform die-off. For example, one day detention at a conservative, exponential decay (die-off) rate of 1 per day would result in 90 percent removal.

2. BOD/DO - The cause of low DO in the lower River is the saline intrusion from Hillsborough Bay. The relatively stagnant saline wedge traps organic materials and allows depletion of DO by sediment oxygen demand. The only method of alleviating this problem would be to inhibit saline intrusion (i.e. construct a barrier) and dredge the bottom sediments. As shown in the model results, some DO depletion may be attributed to stormwater BOD; however this is not the most significant impact.

3. Solids and Metals - Although the solids criteria are currently not violated by stormwater input, solids are responsible for carrying some metals into the lower River. Metal violations cannot be attributed strictly to stormwater, because other sources of metals exist such as the upstream drainage basin and the Bay. Concentrations in the River for certain parameters (i.e. copper and zinc) are sometimes above standards. However, the concentrations are seldom more than twice as high as the criteria except in the case of

mercury. It therefore seems reasonable to reduce solids (and thus metals) loadings to the lower River whenever possible by either removal or detention.

Ranking of Problem Areas. The largest subarea stormwater loadings were indicated in Table 3-4. In order to tie in quality aspects with flooding, Table 3-7 has been prepared. This ranks each subarea giving equal weight to stormwater flooding and quality. For flooding, the seventeen subareas are ranked with respect to flooding complaints as described in Table 2-2 and predicted flooding during the 50 year storm as illustrated in Figure 2-8. For the subareas that do not flood during this event, their rank is assigned based on the activation level of their stormwater system during the event. The rank is from one to 17, with 17 corresponding to the most complaints or flooding. For quality, data presented in Table 3-4 serves as a problem area ranking. The rank is even, from 2 to 34, with 34 corresponding to the worst problem area. (Note that the quality ranking is doubled to give it the same weight as flooding). There are many methods of ranking which may be utilized. This method is intended to serve as an example.

Ranking of Stormwater Control Measures. Based on analysis of stormwater control effectiveness (4), potential stormwater flooding controls are ranked in terms of their ability to reduce stormwater load. In Chapter 2, these were classified as either increased capacity, increased storage, or reduced runoff. Their pollutant removal ranking is shown in Table 3-8, and explained as follows:

TABLE 3-7. RANKING OF PROBLEM AREAS BY FLOODING AND QUALITY

Sub-Area	Predicted flooding rank	Flooding complaint rank	Quality rank	Total Points
TH	13	12	32	57
HA	12	13	28	53
KC	9	16	26	51
SJ	10	7	34	51
SC	16	4	30	50
NW	14	7	24	45
NT	15	5	22	42
WT	11	11	18	40
SP	7	17	14	38
WW	6	15	10	31
RB	4	9	16	29
FH	17	10	2	29
UT	3	1	20	24
OG	2	14	4	20
SS	5	7	6	18
SH	8	2	8	18
DT	1	3	12	16

1. Runoff Reduction - This can be achieved by devices such as drain trenches and ditches and results in a 30 to 70 percent removal rate (hydraulic and pollutant load).

2. Storage - Retention/detention pond removal rates can vary from 10 to 90 percent. They are effective in removal of the particulate phase of pollutants (solids, metals) but not dissolved phases (BOD, nutrients). They attenuate the peak hydraulic load but in general do not result in reductions in volume. The wide range of removal rates is primarily

associated with the amount of dead storage volume that is available during a storm.

3. Capacity - Increased capacity does not result in any reduction in pollutant load. In cases where flooding is alleviated by using increased capacity, increases in pollutant load may occur. Of the types of stormwater management approaches available, this should only be used in instances where other methods are not feasible.

SMP Procedure. In Chapter 2, a procedure was formulated for analysis of stormwater quantity problems. This procedure is expanded to take into account quality issues, as depicted in Figure 3-10. As in Chapter 2, the procedure is divided into a preliminary and detailed planning phase. The preliminary planning phase takes into account the information prepared during the NURP program. The detailed planning phase is a modification of the City's current Master Drainage Planning efforts. The stormwater quality assessment steps and how they fit in with the overall procedure are described below.

Identify Quality Problems and Objectives - The loads presented in Table 3-4 and the subarea rankings in Table 3-7 provide the basis for identification of stormwater quality problem areas. The City may wish to finalize this ranking to provide a priority system for addressing stormwater problems.

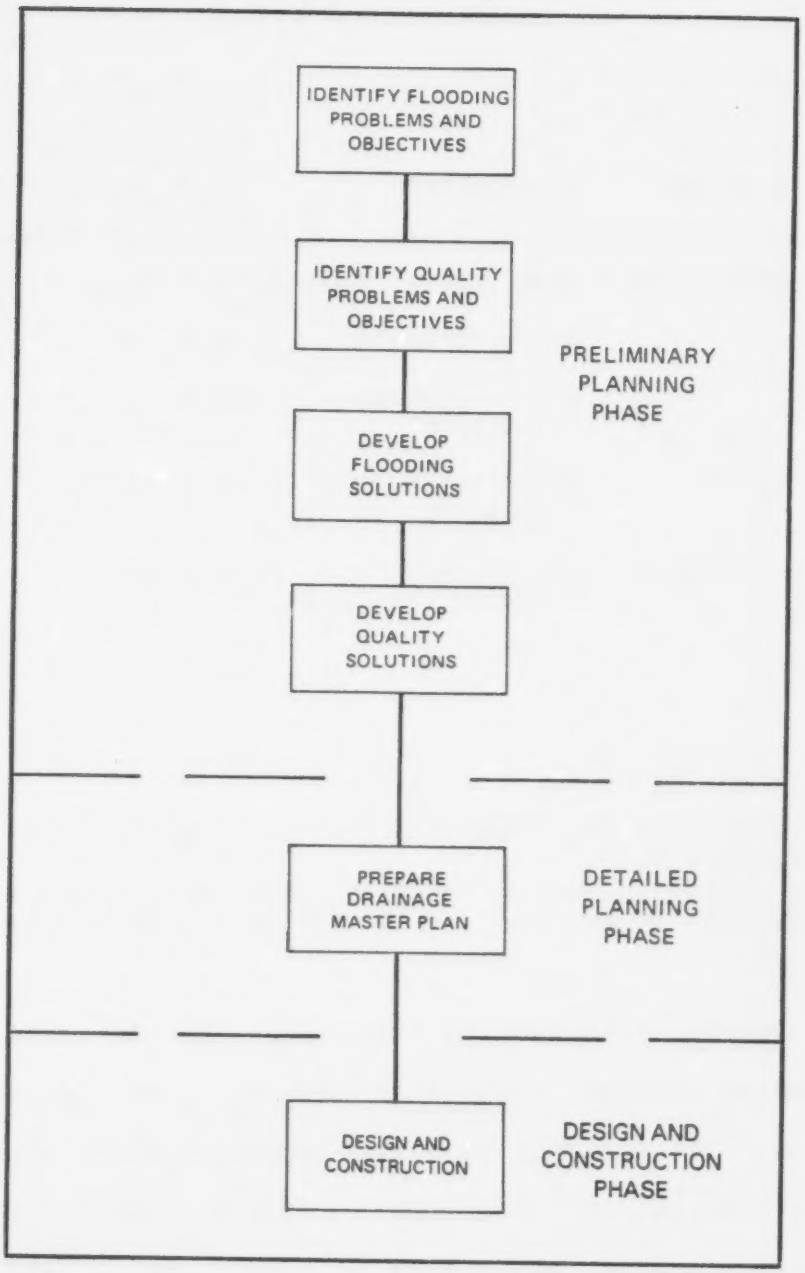


FIG. 3-10 STORMWATER QUANTITY AND QUALITY MANAGEMENT PROCEDURE

TABLE 3-8. SUMMARY OF REMOVAL RATES FOR VARIOUS STORMWATER CONTROLS[1]

Classification	Stormwater Control	Long-term hydraulic removal rate (% of inflow rate)[2]	Long-term pollutant removal rate (% of inflow load)
Runoff Reduction	Open bottom inlets	0 - 5	0 - 5
	Drain trenches	30 - 35	30 - 35
	Ditch systems - regular	30 - 70	30 - 70
	Ditch systems - undercut	50 - 70	50 - 70
Storage Increase	Retention/ Detention ponds	20 - 95	0 - 30 (soluble pollutants) 0 - 90 (particulate matter)
Capacity Increase	Storm Sewers	0	0

1. This Table summarizes work performed during Task II.9, reference 4.
2. The hydraulic removal rate for detention ponds is the percent reduction or attenuation of the peak inflow rate.

Characterize Quality Solutions - During characterization of flooding systems, a range of possibilities was developed for analysis. The analysis took into account needed flow reductions and their costs. To incorporate quality into this assessment, the receiving water objectives as summarized in Table 3-6, and the effectiveness of stormwater controls in Table 3-8 must both be taken into account. For each potential solution, the removal

rates should be determined and compared to the desired objectives. Before proceeding into the detailed planning phase, an evaluation of potential solutions which takes into account the quantity and quality aspects presented in Chapters 2 and 3 should be performed. Such an analysis provides information on the likelihood of success of several alternative solutions, and thus a direction for the detailed planning phase.

Detailed Planning Phase - Discussion of the City's Master Drainage Planning effort was given in Chapter 2. Quality data which may be required for selection of a recommended drainage plan include the quality of storm runoff in the basin and the effectiveness of existing controls. The need for such tests is dependent on the availability of such data in nearby areas. The NURP sampling conducted by the City already provides excellent guidelines for such information. The recommended plan for the basin must conform to the receiving water objectives as summarized in Table 3-6. By following the approach given in Figure 3-10, quality factors will be taken into account in the plan development.

Summary. A procedure has been presented for incorporating both quantity and quality issues into stormwater management planning. This procedure will be expanded in Chapter 4 to take into account the State permitting system for stormwater discharges. In Chapter 5, the methods developed so far will be applied to illustrate their use for a test case basin.

REFERENCES

1. Metcalf & Eddy, Inc., Water Quality Assessment of the Lower Hillsborough River, City of Tampa, Tampa Nationwide Urban Runoff Program Study, October, 1983.

2. Metcalf & Eddy, Inc., Task II.2, Rainfall Quantity Analysis, City of Tampa, Tampa Nationwide Urban Runoff Program Study, February, 1983.

3. Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc., Storm Water Management Model, Volumes I-IV, Environmental Protection Agency, Water Quality Office, 1971.

4. Metcalf & Eddy, Inc., Phase II Report, Task II.6 Runoff Characterization - Data Analysis, Task II.9 Control Testing - Data Analysis, Task II.10 Projection of Study Area-Wide Loadings, City of Tampa, Tampa Nationwide Urban Runoff Program Study, October, 1983.

5. University of Florida, Stormwater Management Model User's Manual Version III, Environmental Protection Agency, Office of Research and Development, 1981.

6. Metcalf & Eddy, Inc., Development and Application of a Simplified Stormwater Management Model, Environmental Protection Agency, Water Quality Office, 1976.

7. Dailey, J. E. and Harleman, D.R.F., Numerical Model for the Prediction of Transient Water Quality in Estuary Networks, Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Report No. 158, Department of Civil Engineering, Mass Institute of Technology, October, 1972.

8. Priede-Sedgwick, Inc., Runoff Characterization - Water Quality and Flow Data, Task II.5, Tampa Nationwide Urban Runoff Program Study, Phase II, March, 1983.

9. U.S. Geological Survey, Water Resources Investigations, 48-125, Hydrologic Data from Urban Watersheds in Tampa Bay Area, Florida, 1979.

CHAPTER 4

STORMWATER PERMITS

General

The State of Florida Administrative Code Chapter 17-25 sets forth a permitting system for stormwater discharges. The permit requires specific levels of stormwater control which result in certain removal or treatment of stormwater flows and loads. The purpose of this section of the Stormwater Management Plan is to evaluate the permit requirements and relate them to the strategies which deal with stormwater flooding and pollution developed in Chapters 2 and 3.

To evaluate the permit with respect to stormwater management planning in the City of Tampa, a description of the regulation is first given. The analysis of stormwater quantity in Chapter 2 and stormwater quality in Chapter 3 are then reviewed in terms of the regulation. The generalized stormwater management planning approach is then revised to incorporate the permitting procedure.

Florida Stormwater Permit Regulation

The major components of Chapter 17-25 are indicated in Table 4-1. A description of the important aspects of the regulation as it relates to stormwater management planning in Tampa follows.

25.01 Scope - The basic reason for the regulation of stormwater discharge by the State is for the protection of

TABLE 4-1. MAJOR COMPONENTS OF FLORIDA STORMWATER DISCHARGE REGULATION

Section	Description
25.01	Scope
25.02	Definitions
25.03	Exemptions
25.04	Construction Permit Requirements for New Stormwater Discharge Facilities
25.05	Delegation
25.06	Relationship to Other Permitting Requirements
25.07	Transferability of Other Chapters
25.08	General Provisions
25.09	List of Entities to Which Permitting Pursuant to this Chapter has been Delegated, Addresses, Delegation Documents and Rules Adopted by Reference
25.10	Effective Date

receiving water quality, as is enforcable through the water quality standards regulations (Chapter 17-3).

25.02 Definitions - Several definitions in the regulations of interest in stormwater management are summarized in Table 4-2. The key definitions establish which facilities must obtain stormwater discharge permits, and define the State concept of stormwater treatment.

25.03 Exemptions - The major exemptions to the permit regulations are given in Table 4-3. Basically the regulation requires retention or detention with filtration of the runoff from one inch of rainfall, or one-half inch of runoff for areas less than 100 acres. The basis for this rule is a rainfall analysis done by the University of Florida which indicates that 90 percent of the storms have less than 1 inch of rainfall. The

TABLE 4-2. SELECTED DEFINITIONS IN THE FLORIDA STORMWATER PERMIT REGULATIONS[1]

Stormwater Discharge Facility	A stormwater management system (i.e. design features which collect, convey, channel, hold, inhibit or divert stormwater) which discharges to State waters.
New Stormwater Discharge Facility	A stormwater discharge facility which was not in existence as of February 1, 1982 (or for which no permit application has been received), or an existing facility which is to be modified.
Detention	Temporary storage of stormwater with a 72 hour detention time.
Retention	Complete removal of stormwater by soil percolation, evaporation or evapotranspiration within 72 hours.
Filtration	Removal of suspended matter from stormwater by passing through suitable fine textured granular material.
Grassed Waterway	A trench of certain design characteristics which contains standing or flowing water only after storm events.

1. These definitions are currently being reviewed and modified.

State is implicitly requiring removal or treatment of 90 percent of the storm events.

25.04 Permit Requirements - If the Applicant can demonstrate conformance to one of the conditions in Table 4-3, the State will issue the permit. If not, the State will consider the following factors:

1. whether best management practices are used,
2. whether the public interest is served,
3. the efficiency and cost of alternative controls,

TABLE 4-3. EXEMPTIONS TO STORMWATER PERMIT REGULATION(1)

1.	One single family dwelling, duplex, triplex or quadriplex which is not part of a larger development.
2.	Residential projects of less than 10 acres total area with less than 2 acres impervious surface, provided that it is part of an approved master drainage plan.
3.	Facilities consisting of swales.
4.	Facilities for agricultural land.
5.	Facilities for silvicultural land.
6.(2)	Facilities which provide retention, or detention with filtration, of the runoff from the first one inch of rainfall, or from the first one-half inch of runoff for areas less than 100 acres.
7.(2)	Facilities consisting of ditches/swales designed to percolate runoff from a 3-year 1-hour design storm within 72 hours.
8.(2)	Modification or reconstruction of an existing system which will not serve new development, and which will not increase pollutant load.

1. These exemptions are currently being reviewed and modified.
2. Subject to approval by an engineer, registered in Florida, before construction.

4. whether water quality benefits are reasonably related to costs, and

5. whether reasonable operations and maintenance provisions have been made.

Impact of Permit on Stormwater Quantity

The major permitting issues which relate to stormwater quantity are:

1. The effect of retention or detention of runoff from 1 inch of rainfall.
2. The effect of 72 hours of detention or retention time as a basis for design.
3. The impact of a 3-year 1-hour design storm for design of ditch/swale systems.

Each of these issues is evaluated with respect to results of the NURP Study.

Retention of Runoff Volume. As mentioned, one of the bases for requiring retention of the runoff from the first inch of rainfall was an analysis which concluded that 90 percent of the storms in Florida had less than one inch of rainfall. Therefore, capture (retention) or treatment (detention with filtration) would presumably remove or treat 90 percent of all storm events.

Out of a total of 2,211 storm events that were analyzed during the Tampa NURP Rainfall Quantity Analysis (1), 294 (13 percent) have a rainfall of 1 inch or greater. However, these larger storms contribute about 55 percent of the total rainfall. Assuming that the first inch of rainfall from these large storms is treated, the remaining untreated rainfall volume would represent 26 percent of the total rainfall. Therefore, although 87 percent of the storms would be contained, the actual capture or treatment of rainfall (required by Chapter 17-25) is about 74 percent. This is because the larger storms have proportionately more rainfall and runoff.

The City of Tampa specifies that detention or retention ponds be designed based on 25 or 50 year frequency storms (over 10 inches of rainfall). Because of this, the permit requirement of retention of runoff from the first inch of rainfall is less stringent. However, very few ponds are currently being built in Tampa, as the City does not require them in most cases. Also, because of the high groundwater elevations in Tampa, it is difficult to build ponds of sufficient size and capacity. In addition, the performance of these ponds varies seasonally based on groundwater table elevation and magnitude of the storm. Despite these factors, when retention ponds are built to the City's design specifications, compliance with the State permit requirements will be more than adequately met.

Pond Drawdown Time. The basis of the requirement of 72 hours detention time is not clear, although it may be related to mosquito control. The rainfall analysis performed during the Tampa NURP Phase II (1) Study can be used to evaluate the use of this criterion in Tampa.

Characteristics of rainfall patterns in Tampa are such that in many cases more than one storm event occurs within 72 hours of each other. The mean separation time between storm events in Tampa is about 68 hours. This is quite close to the 72 hour value. Thus, the 72 hour time period may be thought of as the average time required for drawdown before the next storm event occurs. However, this also indicates that for about one half of the storm events, the effect of a previous event may not

have been dissipated within the specified 72 hours, depending on the size of the event.

During the wet season in Tampa, short duration, high intensity rainfalls are the most frequently occurring type of storm. These storms often occur on a daily basis. During this period, ponds with a 72 hour drawdown time are expected to function far less effectively. Thus, permit requirements regarding retention time are not conservative. During the rainy season, it is expected that under most conditions the storage volume would be partially filled. The average wet season separation time is about 24 hours and the average wet season rainfall is about 0.5 inches (1). For these conditions, the pond volume would be 67 percent filled on the average.

Design Storm for Ditches/Swales. Percolation of the runoff from a 3-year 1-hour design storm within 72 hours is required for ditch/swale systems. Based on the rainfall analysis, a 3-year 1-hour design storm has about 2 inches of rainfall. Hydraulic removal rates of 30 to 70 percent were attained during testing of ditches (2). Undercut ditches were found to remove 50 to 70 percent of the inflow. Controls were not tested for rainfalls greater than about 1 inch. The additional rainfall of 1 inch in the 3-year 1-hour storm would have some significant impact on test results, possibly slight reduction in removal rate.

Since ditches remove 30 to 70 percent of the runoff, then they are from 30 to 70 percent effective in meeting permit

requirements. Most existing ditch systems do not conform to the permit requirement. Based on field observations, percolation of runoff in the ditches occurs in a far more rapid time interval than 72 hours. Rapid percolation was observed even during wet antecedent conditions. Therefore, if ditches are designed to include sufficient storage to retain the remaining 30 to 70 percent of the runoff volume, they would be able to meet the permit requirement.

Impact of Permit on Stormwater Quality

The three major permitting requirements which were analyzed for their effect on stormwater quantity are reviewed in this section for their impact on quality.

Retention of Runoff Volume. The permit requirement for retention of the runoff from one inch of rainfall would result in removal of about 74 percent of the total volume of stormwater. Therefore, pollutant removals of at least 74 percent could be expected. For ponds designed by the more stringent City standards, additional removal could be expected. However, as mentioned, such ponds are not often constructed. For the flow which is not removed and does enter the receiving water, some removal of particulates and metals can be expected, as discussed in the Tampa NURP Phase II (2) report. The removal of bacteria by the ponds was not analyzed in Phase II, but assuming a die-off rate of 1 per day, about 90 percent coliform removal per day of detention can be assumed. The effect of compliance with the permit rule is therefore a minimum of 74 percent removal when the pond is operating correctly.

Pond Drawdown Time. As shown, the separation time between storms indicates that the ponds will often be partially full during the wet season. This would result in less removal of pollutant loading. As mentioned in the stormwater quantity discussion, the pond will be about 67 percent full on the average during the wet season. This would decrease the stated removal (74 percent) to around 25 percent. Pollutant removals would therefore be a minimum of 25 percent during this condition. If the ponds are sized according to City design requirements rather than State permit requirements, somewhat better removal would be obtained because the ponds would be dry more frequently.

Ditches/Swales. Pollutant removal for ditches is assessed based on hydraulic removal. If ditches are designed as specified in the permit, complete removal of flow and thus pollutant load should be achieved under all but the most severe conditions. This is because of the rapid infiltration observed in these ditches.

Summary. The effectiveness of various controls was summarized in Table 3-8. This information can be used to determine expected hydraulic and pollutant removal rates under DER stormwater permit conditions. However, in order to account for variability in pond effectiveness, pond pollutant load removal must be separated into two components: that which occurs together with flow retainment (equal to percentage flow reduction) and that which occurs from flow detainment. Table 4-4, as obtained from the Tampa Phase II Study(2), presents

the percent range of pond pollutant removal due to flow detainment.

TABLE 4-4. POND POLLUTANT REMOVAL RATE DUE TO FLOW DETAINMENT

Pollutant Type	Pollutant Removal Rate (percent)
Soluble Pollutants	0-30
Particulate Matter	0-90

With this information, the expected hydraulic and pollutant removal rates under DER stormwater permit conditions of Table 4-5 were calculated. Flow reductions in ponds during the dry season are based on the expected capture of Tampa rainfall and the assumption that 72 hours is always available for drawdown. During the wet season, a separation time of about 24 hours between storms is expected to cause the ponds to be about 67 percent full, or 33 percent effective. These values for effectiveness are combined with the 74 percent capture of all rainfall developed earlier in this Chapter to arrive at the calculated flow reductions.

Long term flow removals are based on a weighted average of dry and wet season conditions, with the wet season being one quarter of the year. Ponds are still fairly effective in stormwater treatment since additional pollutant removal, beyond the available storage, can be achieved. These additional removals are those listed in Table 4-4, as discussed.

TABLE 4-5. SUMMARY OF DER PERMIT EFFECTS ON STORMWATER FLOW AND LOAD REDUCTION

Stormwater Permit Requirement	Flow Reduction (percent)	Load Reduction (percent)
Runoff retention with 72 hour drawdown		
dry condition[1]		
soluble pollutants	74	74-82
particulate matter	74	74-97
wet condition[2]		
soluble pollutants	25	25-48
particulate matter	25	25-93
long-term average condition[3]		
soluble pollutants	62	62-74
particulate matter	62	62-96
Ditch[4]	100	100
Undercut ditch[4]	100	100

1. Separation time greater than 72 hours (typical dry season).
2. Separation time about 24 hours (typical wet season).
3. Assumes wet season condition for 25 percent of time (33 percent effective) and dry season for 75 percent of time (fully effective).
4. Pollutant removal assumed equivalent to hydraulic removal.

It must be remembered that the ponds should be designed so that standing groundwater does not occur above the design low water level. As mentioned, ditches/swales would be highly effective in reducing stormwater flows and loads.

Stormwater Management Planning Including Permitting Procedure

In Figure 4-1, the SMP procedure incorporating permit requirements is shown. Each step in the process which involves the permit requirements is described below.

Preliminary Planning; Evaluate Permit Impacts - For the range of flooding and quality solutions developed in Chapters 2 and 3, the effect of implementation of the stormwater permit regulations, as required by the City of Tampa, can be identified using the information presented in Table 4-4.

Detailed Planning; Incorporate Permit Requirements - During Master Drainage Planning, the practicality of implementing controls which conform to DER permit requirements must be assessed. Final flow and pollutant rates based on the selected plan can be determined. If it is technically impractical and economically prohibitive to meet all requirements of the permit, then, only as a last resort solution, the following factors as put forth in the permit should be evaluated and presented to the State:

1. use of best management practices,
2. whether public interest is served (i.e., flood protection),
3. control efficiency and costs,
4. water quality benefits as related to costs, and
5. operations and maintenance provisions

Most of the items listed above would have been evaluated as part of the preliminary and detailed plannning steps. This readily

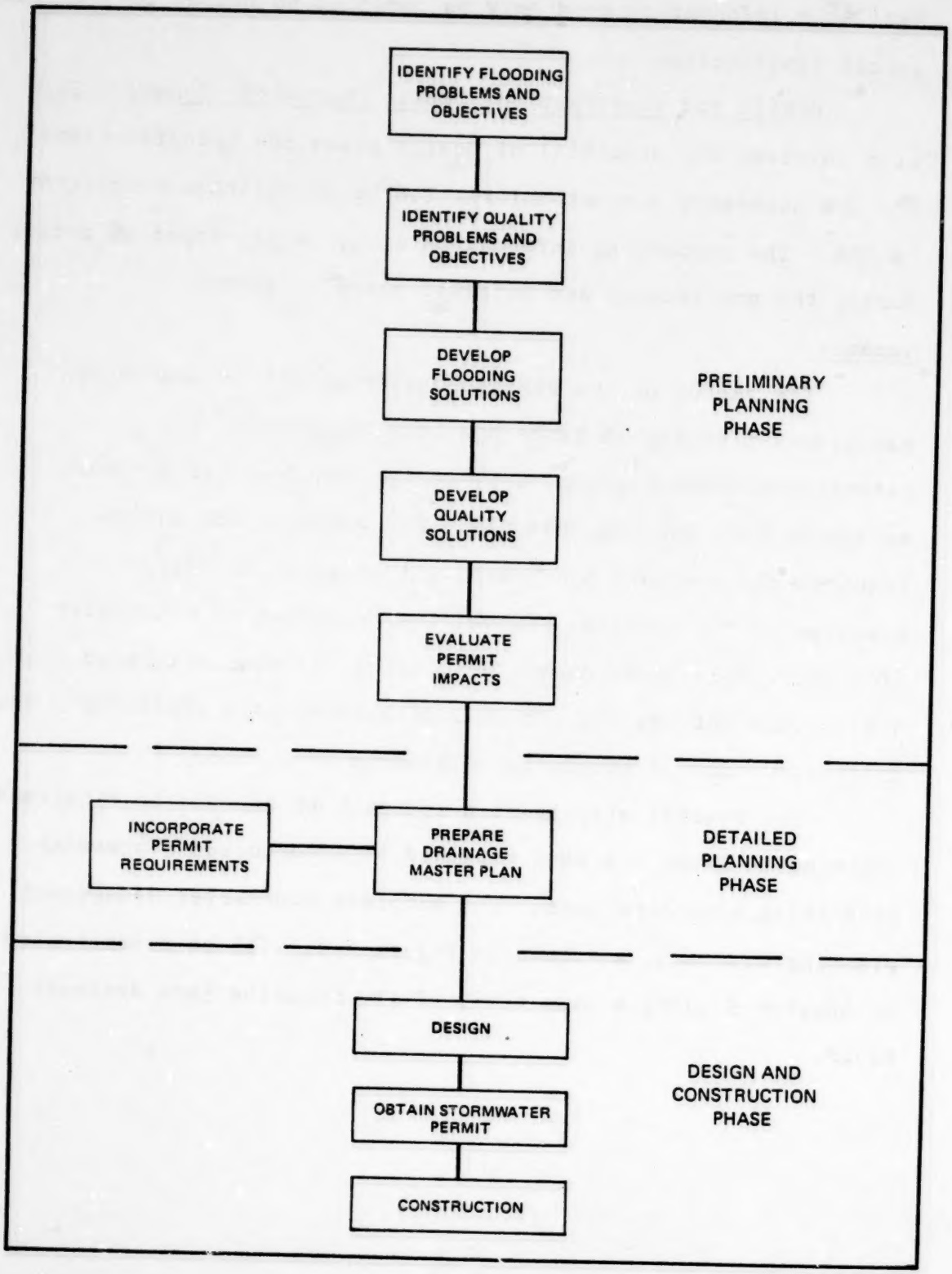


FIG. 4-1 STORMWATER MANAGEMENT PROCEDURE

available information need only be compiled to become part of the permit application.

Design and Construction; Obtain Stormwater Permit - This step involves the submittal of design plans and specifications and the necessary support information to demonstrate compliance to DER. The supporting information would be developed in detail during the preliminary and detailed planning phases.

Summary

The impact of the DER stormwater permit in stormwater management planning in Tampa has been evaluated. For retention/detention ponds, City design requirements are more stringent than DER requirements. For ditches, DER design requirements are more conservative than existing design practices. The expected removal and treatment of stormwater flows and loads under permit conditions has been evaluated (Table 4-4). This information can be incorporated into preliminary and detailed design of stormwater discharge facilities.

The overall step-by-step approach to stormwater management planning in Tampa has been expanded to incorporate stormwater permitting considerations. The complete stormwater management planning approach, as shown in Figure 4-1, will be demonstrated in Chapter 5 using a case study of the Sunshine Park drainage basin.

REFERENCES

1. Metcalf & Eddy, Inc., Task II.2, Rainfall Quantity Analysis, City of Tampa, Tampa Nationwide Urban Runoff Program Study, Phase II, February, 1983.

2. Metcalf & Eddy, Inc., Task II.6, Runoff Characterization - Data Analysis, Task II.9 Control Testing - Data Analysis, Task II.10 Projection of Study Area-Wide Loadings, City of Tampa, Tampa Nationwide Urban Runoff Program Study, Phase II, October, 1983.

CHAPTER 5

CASE STUDY - SUNSHINE PARK

General

In this Chapter, a detailed Stormwater Management Plan (SMP) is developed for the Sunshine Park test case area, which is a subsection of the Sunshine Park subarea described in Chapter 2. The procedure used in preparing this detailed plan can be followed to develop detailed plans for the remaining basins. The development of an SMP for the Sunshine Park subarea follows the procedure developed in previous Chapters. It includes the following three phases:

A. Preliminary Planning Phase

1. Identify flooding problems,
2. Identify needed pollutant load reductions,
3. Develop flooding solutions,
4. Develop quality solutions,
5. Evaluate permit impacts.

B. Detailed Planning Phase

6. Prepare Drainage Master Plan incorporating permit requirements.

C. Design and Construction Phase

7. Design,
8. Obtain stormwater permit,
9. Construction.

The Preliminary Planning Phase involves the application of the techniques developed during the NURP study. Those steps feed into the detailed planning currently being accomplished by the City through Master Drainage Planning. The results of the Tampa NURP study are also useful for the City's Master Drainage Planning effort, particularly the coefficients developed in the detailed stormwater modeling in Phase III. Demonstration of the preliminary and final planning phases of this procedure for Sunshine Park as described in this Chapter will serve as a model for future City efforts.

Test Case Area Description

As indicated in Figure 5-1, the test case area is located to the east of the lower Hillsborough River within the Sunshine Park subarea. It is approximately 436 acres in size. Table 5-1 summarizes projected land usage in the test case area as determined from the Tampa 2000 Land Use Element Maps. Residential land use is projected to remain the primary usage. Based on aerial photographs, it is estimated that 40 percent of the test case area is impervious. Relief in the test case area is generally mild except at the riverbank and on the manmade berm of Interstate 275. A representative ground slope is about 1 ft./100 ft.

Three storm sewer outfalls lead to the lower Hillsborough River from the test case area. There are two, 54 inch pipes off of Violet Street at an invert elevation of -0.6 ft. MSL and one, 60 inch pipe off of South Avenue at an invert elevation of

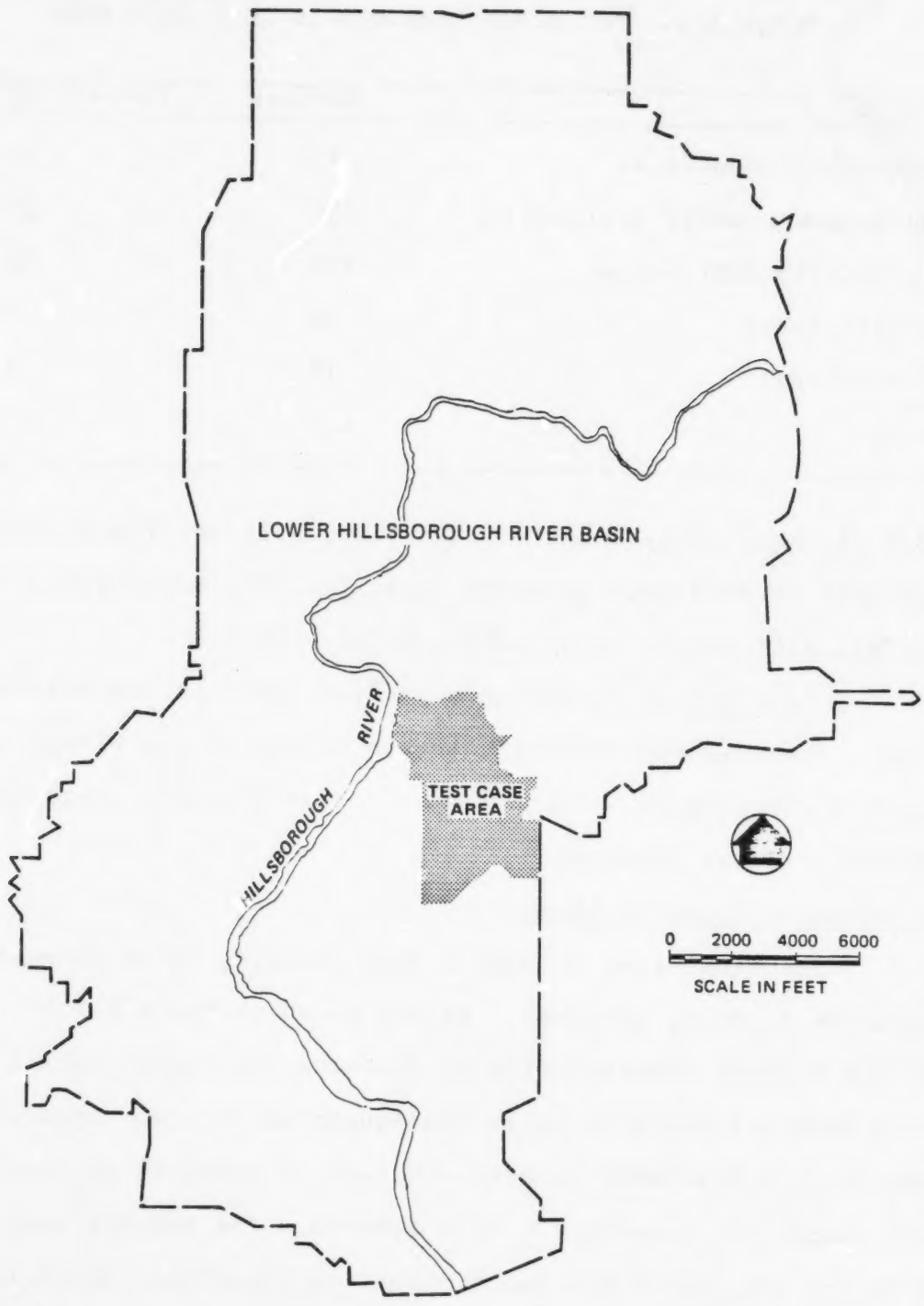


FIG. 5-1 LOCATION OF TEST CASE AREA

TABLE 5-1. PROJECTED LAND USE IN TEST CASE AREA

Land use	Acreage	Percent of total
Commercial/Industrial	87	20
Low/Medium Density Residential	135	31
Low Density Residential	157	36
Institutional	39	9
Open/Natural	18	4
Total	436	100

-3.3 ft. MSL. Only under extreme tides will the Violet Street outfalls be submerged reducing capacity. The South Avenue outfall will generally be submerged at high tide.

There are no retention/detention ponds in the test case area. The primary available storage is the volume within the storm sewer network. Ditches that follow the Interstate provide some additional storage.

Preliminary Planning Phase

The first step in preliminary planning is to identify the existing flooding problems. As was shown in Table 2-2 of Chapter 2, the highest concentration of flooding complaints during 1982 that were attributable to an inadequate storm sewer system occurred in the Sunshine Park subarea. A total of 20 percent of the complaints occurred in this subarea. The primary complaint area was the Violet and South Street neighborhood, which is in the test case area.

Needed pollutant load reductions can be developed from information in Chapter 3 on stormwater quality management. As shown in Table 3-7, the Sunshine Park basin is not a major contributor of pollutant load. However, because of the extensive flooding problems, it has been ranked high on the City's priority list. Load reduction objectives for this subarea are to remove or detain stormwater, in compliance with the State stormwater permitting rule. Therefore, the pollutant control priorities for this subarea are as follows:

Priority	Stormwater Control	Load Reduction
first	retention/detention ponds	65 - 96
second	ditches/drain trenches	30 - 70
last	storm sewer laterals	0

After identification of flooding problems and desired pollutant load reductions, preliminary planning solutions for flooding and pollutant load reductions may be developed. The flood solution curve (Figure 2-10) was used to estimate the magnitude of flooding problems in the test case area and to provide guidelines to flood mitigation requirements. The general flooding solution, as described in Chapter 2, requires that an area's size, runoff coefficient (Rc), outflow capacity and storage volume be specified. For the test case area, these values are:

Size = 436 acres

Rc = 0.5 (based on projected land use)

Outflow Capacity = 9.6 mgal/hr

Storage Volume = 1.7 mgal

Using these values, the no-action condition is plotted on the flood solution curve in Figure 5-2. This Figure shows an enlargement of the section of the five year design storm flood solution curve germane to the test case area. The no-action condition lies to the left of the curve suggesting that flooding will occur during this design event. Following the procedures described in Chapter 2, it can be deduced that any of the following single solutions will alleviate flooding:

1. Increase storage volume by 1.5 mgal (88 percent of the existing value).
2. Increase outflow capacity by 1.7 mgal/hr (18 percent of the existing value).
3. Reduce runoff by 6.8 mgal (14 percent of the projected value).

These single solutions are shown on Figure 5-2. Many combinations of these strategies would also alleviate flooding in the test case area.

Solutions 1 and 3 above will mitigate the impact of pollution from stormwater from the test case area on the lower Hillsborough River. Solution 1 would entail the installation of detention and/or retention ponds. These can be expected to remove part of the pollutant load through the settling of solids.

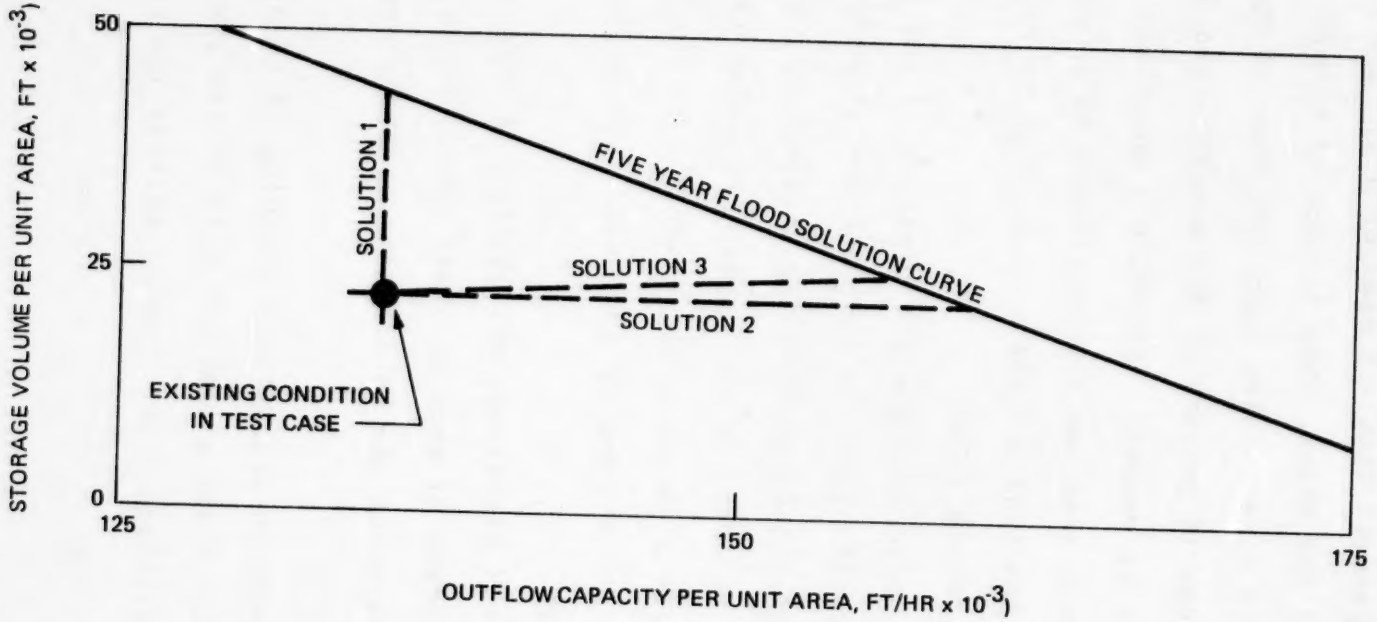


FIG. 5-2 FIVE YEAR FLOOD SOLUTION CURVE APPLICATION TO TEST CASE AREA

METCALF & EDDY

It is difficult to predict how much load reduction will occur due to these ponds because their exact placement within the test case area will determine how much stormwater will flow through them. However, if it is assumed that 1.5 mgal/hr will flow through them for the duration of the storm, then 12 mgal of runoff will be treated. Assuming a conservative load reduction of 65 percent for this volume, then 16 percent of the overall load from the test case area will be removed. Solution 3 would entail the installation of drain trenches and/or ditches to a level at which stormwater runoff and thus pollutant load would be reduced by 14 percent of the projected value.

Solution 2 would increase the capacity of the storm sewer system, and thus would appear to increase the stormwater load to the River. However the major flooding problem area in the test basin is directly adjacent to the River. Flooding from this neighborhood is likely to reach the River; thus, Solution 2 is expected to alter the stormwater pollution load to the River only slightly, if at all.

Following the guidelines established in Chapter 2, the requirements and costs of each of these solutions in terms of the implementation practices can be estimated. These are given in Table 5-2.

The last step in preliminary planning is to evaluate the implications of the State stormwater rule on the preliminary alternatives. Solutions 1 and 2 would satisfy the rule.

TABLE 5-2. PRELIMINARY REQUIREMENTS AND COSTS OF ALTERNATIVES

Alternative	Requirement	Cost ($)
Solution 1		
Retention/Detention Ponds	1.5 acres(1)	94,000
Solution 2		
Drain Trenches	200(2)	634,000
Ditches	30,500 feet	174,000
Solution 3		
Storm Sewer Works:		310,000
Laterals	4,000 ft of 36 in.	
Manholes	10(3)	
Inlets	10(3)	

1. Assumes 3 foot storage depth.
2. Assumes 2, 75 foot lengths per drain trench.
3. Assumes 1 manhole and 1 inlet every 400 feet.

However, each of them has practical problems. The acreage required for the pond alternative may not be readily available in the predominantly residential test case area. Land purchases may require displacing residents. It is clearly desirable to avoid such a policy given reasonable alternatives. The drain trench alternative requires 200 manholes to retrofit the 200 trenches to. However, the test case area does not have 200 manholes, precluding complete implementation of this alternative. Ditches are generally considered a nuisance by those who live near them. Installing 31,000 feet of them in a thickly settled area would meet staunch public resistance.

Solution 3, storm sewer works, does not satisfy the stormwater rule. However, detailed planning may be able to

demonstrate to the State that this alternative is the most reasonable. It is not technically limited and will not disrupt the immediate community.

Detailed Planning Phase

Once the Preliminary Planning Phase has been carried out, and the range of available solutions determined, detailed evaluations are conducted. This can be accomplished using the SWMM.

In order to analyze the test case area using SWMM, it was discretized into 41 subcatchments ranging in size from 1.8 to 30.9 acres. This discretization reflects existing drainage patterns of the area defined by ground slope, surface development and the storm sewer network. Preparation of these model inputs was based on the City's detailed sewer maps. The discretization of the drainage system is shown in Figure 5-3.

The physical characteristics affecting runoff for each subcatchment were defined for input into SWMM. This process included defining the total area and shape, percent imperviousness, ground slope, surface resistance factors, surface storage factors and infiltration rates for each subcatchment. The latter three variables were set to the default values as determined in Phase II and given in Appendix A. The former variables are listed for each subcatchment in Table 5-3.

The runoff from each subcatchment enters the collection system through inlet manholes. The physical description of the

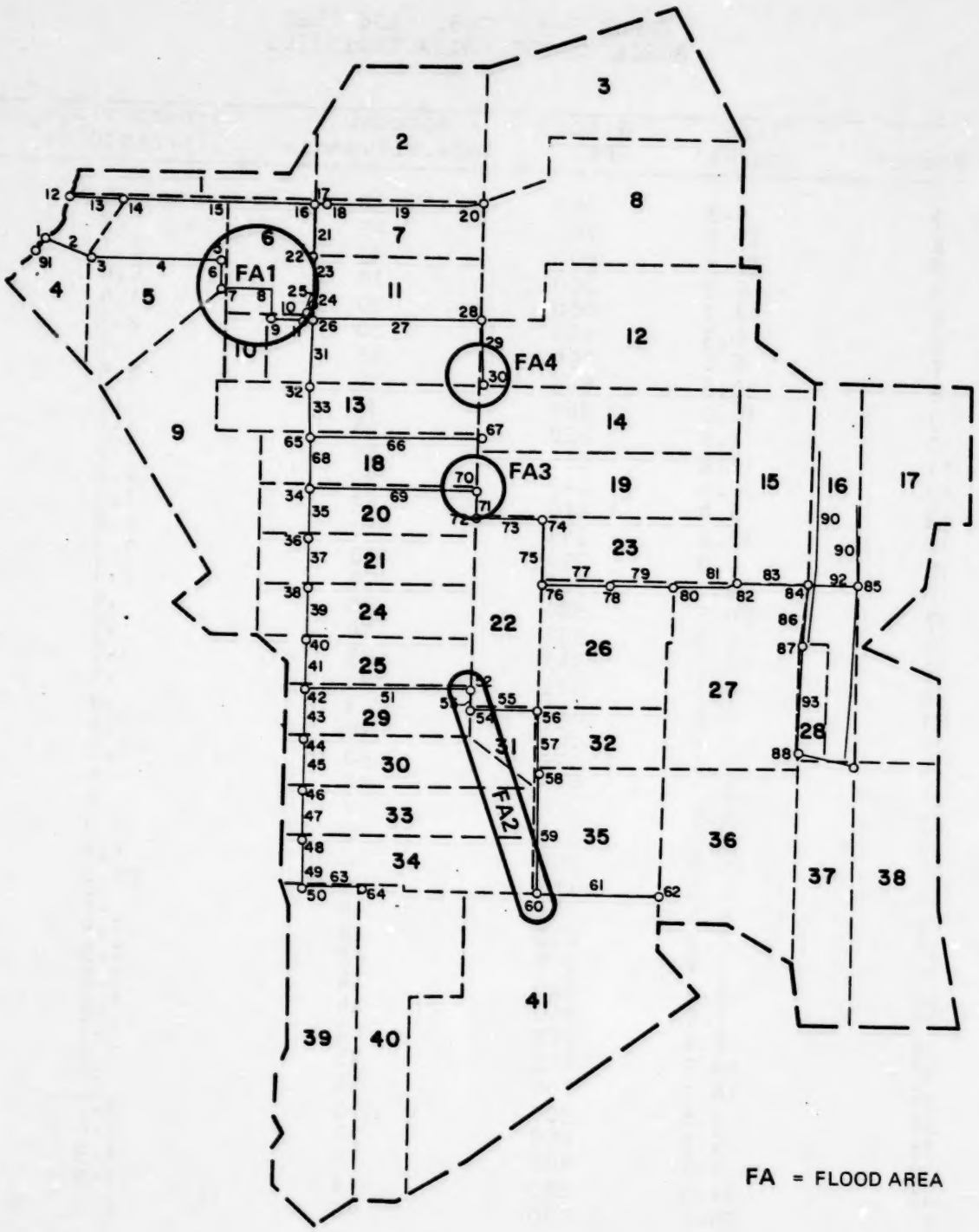


FIG. 5-3 TEST CASE AREA DISCRETIZATION AND STORM SEWER ELEMENTS WITH EXISTING FLOOD AREAS PREDICTED BY SWMM

TABLE 5-3. TEST CASE AREA SUBCATCHMENT CHARACTERISTICS

Number	Size (acres)	Width (ft)	Percent imperviousness	Ground slope ($ft/ft \times 10^{-2}$)
1	4.3	150	30	1.5
2	13.6	705	45	.4
3	17.1	500	45	2.1
4	9.7	800	10	4.0
5	12.1	800	20	1.8
6	6.3	600	30	2.3
7	6.0	295	45	.5
8	20.6	665	30	2.5
9	15.0	365	30	.4
10	1.8	220	20	1.7
11	15.2	655	45	1.0
12	20.4	645	30	2.2
13	8.4	1,375	45	1.3
14	10.8	345	20	1.6
15	9.0	390	30	3.3
16	10.1	3,370	100	7.1
17	20.3	880	30	.5
18	7.3	555	44	.7
19	10.8	345	23	1.5
20	6.8	525	44	.4
21	6.8	520	44	.4
22	8.2	365	30	1.5
23	7.8	335	30	1.7
24	7.3	560	44	.5
25	5.9	510	44	.6
26	10.5	675	30	2.0
27	15.9	700	44	1.4
28	2.0	145	100	20.0
29	5.9	265	44	.9
30	6.9	270	44	1.4
31	2.4	355	44	2.3
32	4.9	320	44	1.0
33	7.9	260	44	1.2
34	8.4	275	44	1.1
35	10.3	675	44	1.0
36	14.1	700	44	.4
37	9.8	305	100	10.0
38	15.5	455	30	.4
39	14.6	380	44	1.3
40	14.3	280	44	1.0
41	30.9	1,200	44	1.0

collection system includes the size, slope and roughness of conduits, the surcharge limit of upstream manholes and estimation of flow distribution at manholes with two downstream conduits. Generally, all pipes 24 inches and larger were modeled. In instances of pipes running in parallel, equivalent capacity single pipes were modeled. The depth of cover ranged from 2.5 to 11.5 feet. Outlet flow distribution between two downstream conduits from one manhole was determined as a function of conduit capacities. Pertinent conduit characteristics are listed in Table 5-4.

The design storm hyetograph in test case simulations is shown in Figure 5-4. This hyetograph was determined from the 5 year design storm curve using a 20 minute interval rather than the 1 hour interval used for flood projections over the entire lower Hillsborough River drainage basin. This time interval was selected so that peak rainfall intensities would accurately represent local flooding conditions. Representing peak rainfall intensities correctly is important as experience in Tampa has suggested that flooding characteristically accompanies storms with high, short duration intensities as opposed to moderate, long duration intensities. In a recent study in Tampa (1), data were collected for thirteen storms with a maximum hourly intensity of greater than 1.0 inches/hour. For these storms the ratio of the maximum fifteen minute intensity to the maximum hourly intensity was 1.8. The five year curve of Figure 2-2 has a

TABLE 5-4. TEST CASE AREA CONDUIT CHARACTERISTICS

Element number	Diameter (ft)	Length (ft)	Slope ($ft/ft \times 10^{-2}$)	Surcharge limit (ft)(1)	Drop (ft)(2)
2	5.8	240	.40	5.0	
4	5.8	700	.70	2.5	.3
6	5.8	150	.48	4.0	
8	5.8	390	.48	3.6	
10	5.8	220	.41	3.0	
13	5.0	240	.64	3.0	
15	5.0	985	.54	11.4	
17	3.0	50	5.50	7.0	3.4
19	3.0	810	.48	4.0	2.0
21	5.0	280	.44	9.0	.4
23	5.0	310	.40	3.0	
27	3.0	890	.81	5.6	1.9
29	2.0	380	.25	5.6	3.4
31	6.9	350	.83	3.6	
33	6.8	270	.83	5.0	
35	5.5	270	.49	4.5	.2
37	5.3	255	.47	4.0	
39	5.3	275	.37	4.0	
41	4.9	255	.44	3.0	.5
43	4.0	265	.36	3.0	.4
45	4.0	270	.40	3.0	
47	3.4	250	.56	3.0	.5
49	3.4	250	.56	3.0	
51	3.5	880	.45	6.0	.4
53	3.0	100	1.00	4.0	2.2
55	3.0	120	.73	6.5	
57	3.0	330	.71	6.5	2.1
59	3.0	640	.29	4.5	1.0
61	2.5	630	.53	5.5	.5
63	2.0	235	1.70	3.0	1.6
66	2.0	880	.50	5.0	
68	5.8	105	.83	4.5	
69	4.9	850	.28	5.4	
71	4.9	150	.93	6.5	
73	4.5	355	.95	5.6	
75	4.5	325	.71	4.9	
77	4.5	350	.75	5.5	
79	4.0	330	.87	6.4	.5
81	4.0	285	.14	6.5	1.0
83	4.0	360	.68	6.0	
86	4.0	370	.43	10.1	
92	3.0	250	1.40	11.1	
93	4.0	535	.39	11.0	
94	4.0	250	.84	7.9	

1. Equivalent to cover above crown at upstream end of conduit.
2. Drop at downstream end of conduit.

METCALF & EDDY

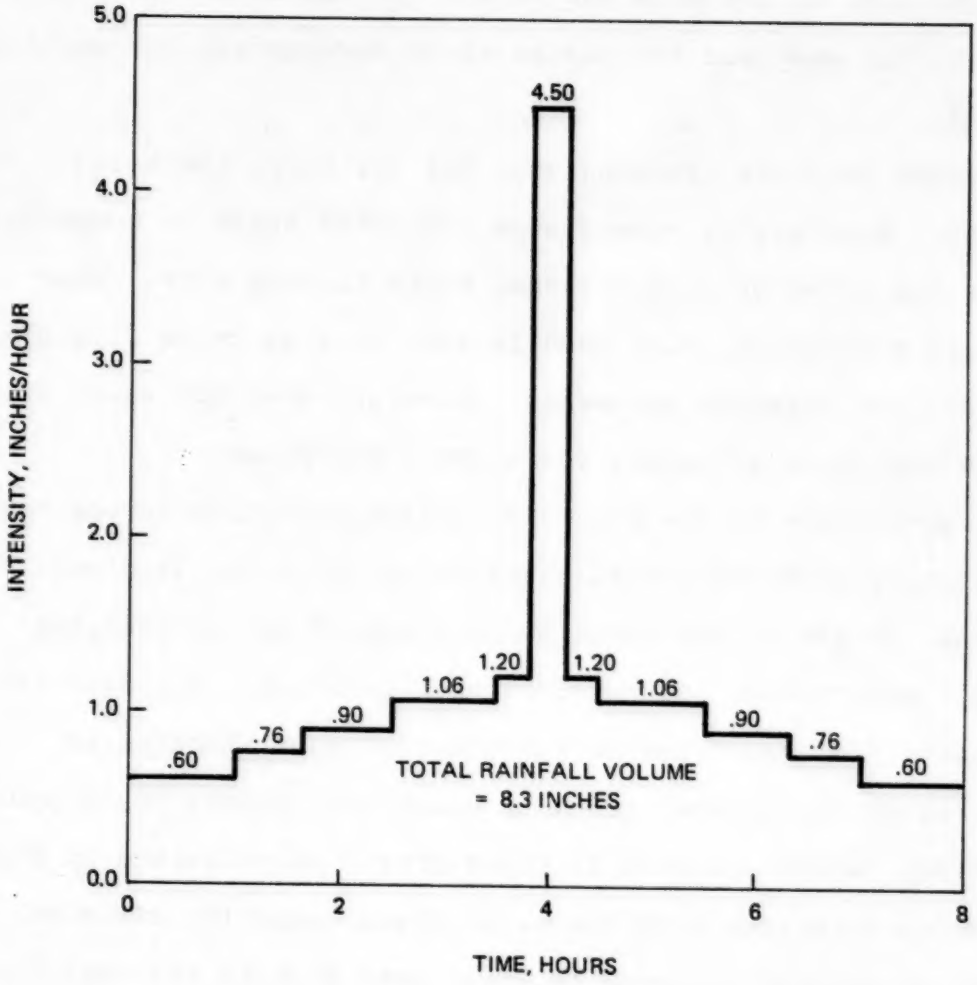


FIG. 5-4 FIVE YEAR, EIGHT HOUR DESIGN STORM HYETOGRAPH USED IN SWMM SIMULATIONS OF TEST CASE

maximum hourly intensity of 2.2 inch/hour. Using a 20 minute interval with this curve gives a maximum intensity of 4.5 inch/hour, which is reasonably consistent with the factor of 1.8, given the size of the data set it was derived from. Thus this time interval was used for design storm development in detailed planning.

SWMM predicts flooding when the surcharge limit is exceeded. Surcharging occurs when the water depth in a manhole exceeds the crown of a downstream, fully flowing pipe. When pipes are surcharged, more head is available to drive flow down the pipe, and capacity increases. However, when the water depth exceeds the depth of cover, the manhole overflows.

Simulation of the future no-action condition during the design storm event caused surcharging in 35 of the 44 pipe elements. Eight of the manholes upstream of the surcharging conduits overflowed. Out of 63.4 mgal of runoff, 5.1 mgal (about 8 percent) flooded. This is 3.4 times as much flooding as predicted by the general flooding solution. Almost 50 percent of this flood volume occurred in Flood Area 1 as indicated in Figure 5-3, which coincides with the major flood complaint district. Another 40 percent occurred in Flood Area 2 while the remainder is fairly evenly distributed between Flood Areas 3 and 4.

Flood control practices simulated in the test case area include detention ponds, storm sewer laterals and drain trenches. Ditches were not simulated as their installation seemed undesirable in a highly residential area. This anaylsis follows the criteria established for the flood prevention

practices in Chapter 2. The practices were implemented in locations consistent with the problem areas identified in SWMM and to initial levels consistent with S/SWMM as developed in the Preliminary Planning Phase and the SWMM no-action simulation.

Detention Pond Solution. Each Flood Area would require a detention pond to completely eliminate flooding. However, for this study, only ponds in Flood Areas 1 and 2, which contribute 90 percent of the flood volume under existing conditions, were simulated. The ponds were initially sized to hold the entire flood volume predicted by SWMM in each Flood Area. The characteristics of the final pond configurations are indicated below. The storage depth required in the ponds was 3.5 and 2.4 feet for Ponds 1 and 2 respectively. Neither pond had any dead storage space available.

TABLE 5-5. CHARACTERISTICS OF DETENTION PONDS SIMULATED IN TEST CASE AREA

Flood Area	Storage Volume (mgal)	Surface Area (acres)	Storage Depth Utilized (feet)	Outflow Gate Width (feet)(1)
1	5.8	5.0	3.5	15
2	1.8	2.2	2.4	5

1. Outflow gates flush with pond bottom and do not act as overflow weirs.

With the inclusion of the ponds during simulation, SWMM predicted no flooding in existing Flood Areas 1 and 2. The conduits in these areas still surcharged. The discharge gate width was sufficient to allow enough outflow to prevent the ponds

from overflowing. These gates were flush with the bottom of the ponds. To totally eliminate pipe surcharging during the 5 year design storm, additional capacity would be required. The storage volume required by this solution was about 5 times that predicted by the general flooding solution.

Storm Sewer Lateral Solution. The storm sewer lateral solution entails the addition of capacity in the pipe networks in the vicinity of the Flood Areas. The characteristics of the additional laterals simulated in each are given in Table 5-6. The increased capacity was determined so that it would accommodate the maximum flood volume out of one manhole in one timestep in each Flood Area predicted by SWMM.

The SWMM simulation of this condition predicted no flooding. While surcharging still occurred in as many pipe elements as the no-action simulation, it did not last as long. It should be noted that the additional capacity required to alleviate flooding as predicted by SWMM is an avarage of 1.4 mgal/hr for the four Flood Areas, which is very consistent with the 1.7 mgal/hr capacity increase projected by the general flooding solution curve. Again, further fine-tuning of model results would be required to totally eliminate surcharging.

Drain Trench Solution. The drain trench solution consistent with the general flooding solution requirements would entail a runoff reduction of 6.8 mgal or 14 percent. It was established in Chapter 2 that one pair of 75 foot long, standard drain trenches servicing one manhole would reduce runoff from one acre by a minimum of 30 percent. This suggests that about 200

TABLE 5-6. CHARACTERISTICS OF STORM SEWER LATERALS SIMULATED IN TEST CASE AREA

Flood Area	Capacity (mgal/hr)	Length (ft)	Location
1	3.0	1780	Lousiana, from intersection with Higland, to Ola to Violet to River
2	1.6	2330	Suwanee, from intersection with Buffalo, to Genesee to Florida to Genesse, at intersection with Highland
3	0.6	1780	Hilda from Highland to Florida
4	0.3	380	Florida from Osborne to Louisiana

drain trench pairs would be required to reduce the runoff from the 436 acre test case area by 14 percent. However, the test case area has less than 200 manholes. Therefore it is proposed to model one drain trench pair per subcatchment inlet manhole. While 41 drain trenches are not expected to completely alleviate flooding, it is close to the maximum feasible usage for this flood prevention practice.

To accomplish drain trench simulation with the SWMM, the Runoff Block (see Appendix A) was used to generate the hydrographs from each subcatchment at the inlet manhole. Each inlet hydrograph was modified by an equation that effectively removed 30 percent of the runoff from one acre of each subcatchment from each inlet hydrograph.

Flooding predicted by SWMM is reduced by approximately 60 percent from the no-action condition. The 2.0 mgal of flood volume still primarily occurs in Flood Areas 1 and 2. Flood Area 3 no longer floods. As was expected from the runoff reduction solution predicted by the general flooding solution, drain trenches alone cannot prevent flooding in the test case area.

Summary. The specific flooding solution analysis demonstrated how the SWMM existing condition simulation and the general flooding solution requirements developed during preliminary planning can be used to formulate flood prevention solutions. Of the three solutions tested, the increased outflow capacity solution performed the best, eliminating flooding altogether. The detention pond solution performed well in the Flood Areas in which it was tested. However the pond acreage required for this solution would be difficult to acquire in the predominantly residential test case area. The drain trench solution performed fairly, reducing flood volume by 60 percent. The installation of drain trenches to flood prevention levels is limited by the number of manholes in the test case area.

Based on comparison of specific flooding solutions, the outflow capacity solution would be the most easily implemented plan. It may be efficacious to introduce some drain trenches into the solution in order to reduce the increased capacity requirements. The specifics of such a combined plan would have to be based on site specific criteria. Standard optimization techniques[3] could be used in conjunction with constraints based

METCALF & EDDY

on site specific criteria to optimize combined solutions. Such factors would be evaluated in detail during Master Drainage Planning.

The outflow capacity solution will not reduce the stormwater pollutant load to the River and will not meet the DER Permit Requirements. If permitting difficulties are encountered, the City could demonstrate its desire to meet the requirements as much as practical by installation of some drain trenches, which will remove some of the stormwater pollutant load.

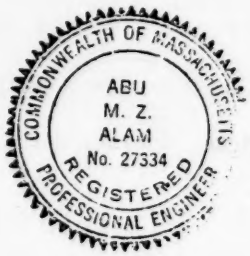
Based on costs presented in Chapter 2, City drainage design criteria and the requirements of Table 5-6, Table 5-7 was compiled. The diameters in this Table were calculated assuming a Manning's n of 0.015 and a pipe slope of 0.004, which is typical of the existing slopes in the Flood Areas.

TABLE 5-7. STORM SEWER WORKS REQUIREMENTS AND COSTS FOR EACH FLOOD AREA

Flood area	Laterals		Number of		Cost for all works ($)
	Diamater (inches)	Length (feet)	Manholes	Inlets	
1	54	1,780	4	4	188,456
2	42	2,330	5	5	157,530
3	30	880	2	2	52,748
4	24	330	1	1	18,454
Totals	-	5,320	12	12	417,188

Conclusions

In this Chapter, the performance of preliminary and detailed planning studies was demonstrated for the Sunshine Park test case area. The method used integrated the S/SWMM flooding solution curve with the SWMM, producing an effective, analytical procedure that can be applied to similar areas throughout Tampa. The SMP included evaluation of plans for the test case area which incorporated stormwater quantity, quality and permitting criteria as developed earlier in this report.



Respectfully submitted,

METCALF & EDDY, INC.

Abu M. Z. Alam
Associate
Registered Professional Engineer
Massachusetts License No. 27334

Approved:

Jekabs P. Vittands
Senior Vice President
Registered Professional Engineer
Florida License No. 27757

REFERENCES

1. Priede-Sedgwick, Inc., Detailed Event Data, Tasks II.5 and II.8, Tampa Nationwide Urban Runoff Program Study, Phase II, April, 1983.

2. Metcalf & Eddy, Inc., Phase II Report, Task II.6 Runoff Characterization - Data Analysis, Task II.9 Control Testing - Data Analysis, Task II.10 Projection of Study Area-Wide Loadings, City of Tampa, Tampa Nationwide Urban Runoff Program Study, October, 1983.

3. Maass, A., Hufschmidt, M.M., Dorfman, R., Thomas, H.A. Jr., Marglin, S.A., Fair, G.M., Design of Water-Resource Systems, Harvard University Press, 1962.